A study of the effect of geological structure and succession on foundation design

Bell, H.A.

University of Durham.

H. A. Bell.

M.Sc. Thesis.

"A STUDY OF THE EFFECT OF GEOLOGICAL STRUCTURE AND SUCCESSION ON FOUNDATION DESIGN."

February, 1966.

INDEX.

Chapter 1.

INTRODUCTION.

Engineering Geology is defined by Nelson in his Dictionary of Mining as "a branch of geological science, forming a link between geology and engineering - particularly civil and mining. It provides a basis of theory to guide engineering practice where earth or rock materials are directly or indirectly involved." Leggat's "Geology and Engineering" quotes Boyd Dawkins, F.R.S., as follows:-

> "Geology stands to (civil) engineering in the same relation as faith to works...... The success or failure of an undertaking depends largely upon the physical conditions which fall within the province of Geology, and the 'works' of the engineer should be based on the 'faith' of the geologist."

This obviously refers to geology as an art. One presumes that the engineer did have supreme Faith in the Art; hence the development of soil and rock mechanics, together with the almost complete control of 'site investigation' by the engineer.

CP.2001 'Site Investigations' specifies the objects of site investigation as follows:-

a. To assess the general suitability of the site for the proposed works.

b. To enable an adequate and economic design to be prepared.

c. To foresee and provide against difficulties that may arise during construction due to ground and local conditions.

d. To investigate the occurrence or causes of all natural or created changes of conditions and results arising therefrom.

Who is then capable of discharging such duties? I would suggest that in Britain neither the graduate Civil Engineer nor Geologist is sufficiently versed to perform such work. Unfortunately the Civil Engineer suffers from too short an acquaintance with Geology, whilst the Geologist usually has little or no knowledge of Engineering principles. With experience each can acquire a working knowledge of each other's field, but usually neither has a sufficient all-embracing general background. No doubt there are a few cases of people eminently qualified in both fields, but these are generally few in number. My impression is that facilities for discussion, whilst being profitable, are no substitute for formal education. It is gratifying to note that at least one University is providing M.Sc. courses in Geotechnical Engineering with applicants from the two fields. Again, however, one is prompted to suggest that approximately eight weeks' training in each other's field prior to commencing the main body of the course is not altogether satisfactory.

Site investigation is usually carried out by specialist contractors who employ senior staff qualified in either geology, civil, or structural engineering. Work however is generally carried out on a competitive basis. Specification and Bills of Quantities prepared by the client form the best basis for equality in competition. However, it is the practice among some Architects and Property Developers, etc., to ask for competitive site investigation not based on any standard of equality; obviously this situation in a competitive market can readily lead to a lowering of standards (a fact which is not always appreciated by the client). Studies of engineering failures usually indicate that site surveys were incomplete or inadequate for the structure. Someone was trying to save money and lost heavily because of it. It is difficult to convince the client of the need for adequate engineering studies as he appears to be getting nothing tangible for his money. The responsibility for soil testing lies with the engineer, architect, or owner; and it therefore lies with the specialist contractor to state precisely in his report if he is not satisfied with the extent of the survey. Most contractors include a covering clause qualifying the context of their report. One such typical clause reads as follows:-

> "The recommendations made and opinions expressed in this report are based on the boring records, an examination of samples and the results of laboratory tests. No responsibility can be held for conditions which have not been revealed by the boreholes, for example between borehole positions. Whilst the report may express an opinion on the possible configuration of strata, both between borehole positions and below the maximum depth of the investigation, this is for guidance only, and no liability can be accepted for its accuracy."

The geologist naturally finds that his field of activity overlaps with the civil engineer and the mining engineer. It has been the author's experience that when dealing with soils, the accepted practice evolved from Soil Mechanics theory is generally adequate for his needs. However, when rock is encountered it is then that one's support on established practice is removed, and the necessity for a more scientific approach than that quoted in the Codes, etc., is apparent.

The body of this thesis is concerned with describing examples where the author has been aware of his own deficiencies in translating "Geology applied to Engineering" as defined by the Codes of Practice, into practice. These problems have been broadly defined as follows:-

1. It is firstly necessary to review current practice as outlined in the Code dealing with Foundations in so far as it relates to rock. - Chapter 2.
2. Loading tests on piles appear to give the most readily interpretable indication of what loads may be carried by rocks. It is essential in this instance to review broadly pile bearing calculations as applied to soils in an attempt to see if these can be translated to soft rocks and hard rocks. - Chapter 3.
3. Calculation of end bearing capacities of piles on rock. - Chapter 4.
4. The effect of dip and structure on the foundations of engineering structures. - Chapter 5.
5. Effects of weathering. - Chapter 6.
6. Effects of glaciation. - Chapter 7.
7. Problems in igneous areas. - Chapter 8.
8. Problems in areas of old mine workings. - Chapter 9.

Chapter 2.

"REVIEW OF CODES OF PRACTICE DEALING WITH FOUNDATIONS ON ROCK."

The civil engineering Codes of Practice 'Site Investigations' and 'Foundations' provide the basis for engineering interpretation of geological phenomena related to foundation design. Whilst it is largely accepted that certain of the recommendations are now proved conservative (by practical means) some basis of theoretical calculation will be required before extensive revisions are made.

The recommendations relating to Foundations on rock include maximum safe bearing capacities in Table 1 page 28 based solely on the shear strength of the soil irrespective of any settlement that may ensue: and have a safety factor of about 2.

Maximum Safe Bearing Capacities for Horizontal Foundations at 2 ft. depth b.g.l. under vertical static loading (with a safety factor of 2) extract from p.28 of the Code 'Foundations'.

Igneous and gneissic rocks in sound condition.....	100 tons sq.ft.
Massively bedded limestone and hard sandstones....	40 tons sq.ft.
Schists and slates..................................	30 tons sq.ft.
Hard shales, mudstones, and soft sandstones.......	20 tons sq.ft.
Clay shales..	10 tons sq.ft.
Hard solid chalk...................................	6 tons sq.ft.
Thinly bedded limestones and sandstones) Heavily shattered rocks)	To be assessed after inspection.

It is noted that rocks have a high maximum safe bearing capacity except where decomposed, heavily shattered, or steeply dipping. Where penetration exceeds 2 ft. into 'sound rock' bearing capacities may be increased by 20% per additional foot of depth but should not be twice the values given in the Table 1. It is suggested that where pronounced cleavage and bedding planes occur, if the strata are level, then full safe bearing capacity may be assumed, but a reduction should be made if the beds are steeply inclined or shattered.

Other recommendations are made which, whilst being sound in their intention, do not provide any basis for calculation of say: variation of safety factor under conditions of steep dip, jointing, etc.

By comparison, the Code where it deals with soils is more explicit and mathematical in its advice.

It is evident that the broad terms of the Code have led to a wide variety of opinions and it is the author's concern that research be made so that liberal thought is not too much encouraged by conservatism in others. The Symposium on Large Dia. Bored Piles to be held by the Civils in February 1966 may produce evidence pointing to a need for revision of the Code where it deals with rocks (especially soft rocks).

Elwyn E. Seelye in 'Foundations, Design and Practice', quotes the following presumptive safe bearing capacities of supporting soils, adapted from the New York City Building Code 1952 as follows:-

Class 1.	Hard sound rock	60 tons sq.ft.
Class 2.	Medium hard rock	40 tons sq.ft.

He then gives the following Presumptive Safe Bearing Capacities of Rocks:

Type of rock	Bearing capacity suggested by E.E. Seelye	Remarks
<u>Igneous</u> such as trap, granite, basalt, and lava.	20-60 tons sq. ft.	Usually hard. Does not erode or dissolve readily. Subject to cleavage planes and bed planes at all angles.
<u>Sedimentary</u> such as		
Limestone	10-20 tons sq. ft.	Medium hard as in limestone, to soft as in chalks and shales. Subject to dissolving erosion and formation of caves. Soft layers and seams, soft overburden. Bed planes generally horizontal.
Shale	8-10 tons sq. ft.	
Chalk	8 tons sq. ft.	
Coral	8 tons sq. ft.	
Sandstone	10-20 tons sq. ft.	
<u>Metamorphic</u> such as		
Gneiss	20-40 tons sq. ft.	Gneiss and schist have igneous characteristics; slate and marble have sedimentary.
Schist	20-40 tons sq. ft.	
Marble	10-20 tons sq. ft.	
Slate	8 tons sq. ft.	

Chapter 3.

DISCUSSION OF FORMULAE USED IN CALCULATING THE BEARING CAPACITY OF PILES.

One of the author's aims is to attempt to assess the bearing capacity of rocks in conditions where it is possible to correlate design formulae with load tests and hence with the Code of Practice 'Foundations'.

In the case of the shallow foundations (i.e. strip footings and isolated bases, etc.) loading tests are not usually carried out, but in the case of piled foundations it is the practice of certain Consulting Engineers and other Bodies to test piles in excess of the working load and measure accurately the settlement. It would appear that the task of the various workers involved has been to assess bearing capacity of piles in soils. These formulae are discussed below because in view of the lack of design formulae relating to rock it is felt that benefit will be gathered by assessment to see if they can be applied to soft rocks, and uncemented rocks.

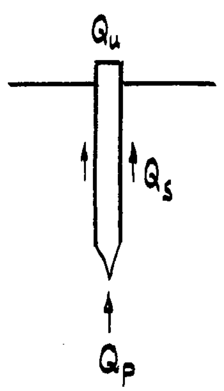
The pile formulae include for a depth factor which increases the ultimate end bearing capacity of a deep foundation (such as a pile) by approximately 50% of that which would be allowed by a shallow foundation. The use of a depth factor has yet to be related to piles founded on rock.

Tomlinson in a paper read at the 'Symposium on large dia. bored piles' September 1961, stated that "where large diameter bored piles are bearing on hard rock such as granite, hard sandstones and massive hard limestones, their allowable carrying capacity is governed by the permissible stress on the concrete in the pile shaft. However, in the case of piles in soft rocks such as marl, chalk, shale and weak sandstone, there is no satisfactory basis for design. The use of arbitrary figures of bearing capacity given in handbooks of Codes of Practice is likely to be uneconomical in cases where piles have to be taken to a considerable depth to reach the rock. The author has heard of a case where a higher working load has been permitted for ordinary precast concrete piles driven to refusal in rock than for large bored piles three or four times their size founded on the same rock. In the first case a dynamic formula was used to calculate the working load; in the second case the allowable bearing pressure was taken from building regulations appropriate to shallow foundations. The development of the science of rock mechanics is needed to enable allowable bearing pressures to be calculated on a realistic basis."

I have been unable to trace any British development of rock mechanics relating to assessment of bearing capacities of rocks under building foundations, but it is clear that this development is necessary if the arbitrary rise of capacities is not to end in failure.

Let us now consider how an assessment of bearing capacities in soils may be made:-

1. <u>The ultimate load</u> on a pile is usually defined as the load at which the settlement exceeds some reasonable value. It is considered in the case of soils to be the sum of two components: that which is due to skin friction on the shaft Qs, and Qp which is due to point resistance.



Ultimate load or bearing capacity

Qu = Qp + Qs

(Terzaghi and Peck 1948)

By calculation of skin friction on piles driven through soils founded on rock, related to working load, the author attempts in Chapter 4 of this thesis to assess the actual load being carried by the rock without undue settlement effects.

2. Design procedure follows three steps: (i) the calculation of the point resistance or end bearing capacity, (ii) the calculation of the skin friction or shaft bearing capacity, and (iii) the working load of the pile is then deduced by applying a factor of safety to the ultimate bearing capacity together with a reduction factor to allow for the inter-action of piles within a group.

(i) To a close approximation the end bearing capacity may be written:

$Qp = Ap.N.Cp.$

where Ap is the area of the base or point, Cp the shear strength of the clay at the level of the point, and N is a bearing capacity factor, which is equal to 9.0 for circular areas loaded at a considerable depth within a saturated clay.

(ii) The shaft bearing capacity is calculated from the expression:

$Qs = As.Ca.$

where As is the area of the shaft of the pile in the clay, and Ca is the average adhesion between the pile and the shaft. Ca can be expressed in terms of the average undisturbed shear strength of the clay, C, thus: $Ca = Xc$

where X is a factor less than unity and depends on the degree of softening of the clay immediately adjacent to the contact surface.

The adhesion is also less than the shear strength of the clay in the case of driven piles (Tomlinson 1957) but the reduction may be attributed partly to lack of contact between the pile and the clay due to whipping of the pile during driving. Expressed as a percentage of the cohesion C, the adhesion for driven piles falls with increasing stiffness of the clay from approximately 100 in very soft clays to 20 in very stiff clays.

(iii) The working load Qw is calculated from the expression:

$$Qw = \frac{B.Qu}{F}$$

where Qu is the ultimate load on an individual pile, B is a group factor and F is the factor of safety. Concerning the group effect, the optimum design spacing to obtain individual action and maximum capacity for the smallest foundation is 1.75 diameters for a two-pile group to 2.5 diameters for a 16-pile group. In addition, B depends on the number of piles and varies between 0.9 for a two-pile group to 0.8 for a 16-pile group. A minimum factor of safety of 2.5 should be used on the calculated ultimate load for piles of the usual dimensions. This will ensure that the settlement is kept within safe limits and also with the inevitable variations from the calculated ultimate load any one pile will be able to pass the acceptance test or proof loading, e.g. the pile should not have a permanent set of more than 0.25 ins. after loading to 1.5 Qw for 24 hours (this figure is apparently accepted by most Engineers).

Determination of the adhesion factor X.

In the case of bored piles, softening of the clay occurs due to an increase in water content of the clay adjacent to the pile and may be caused by:-

(a) Water flowing out of the clay itself during boring from water at the higher level.

(b) Water from the body of the clay due to stress relaxation whilst boring.

(c) Water introduced into the borehole to assist boring operations.

(d) Water absorbed from the concrete which has to be placed at a water-cement ratio greater than that required solely for hydration of the cement. (a) and (c) can be eliminated by good drilling techniques and high rate of boring, but (b) and (d) are inevitable.

With increase in water content, X decreases from a maximum of 80% to 20% at a 6% increase in moisture content, for London Clay.

Meyerhof and Murdock (1953) who based their calculations on the fully softened shear strength derived an expression equivalent to an X value of 0.3. On the other hand, Golder and Leonard (1954) indicated an upper value of 0.7 for London Clay under good site conditions.

Although a considerable volume of data appears to substantiate the Meyerhof theory, in many cases the observed "ultimate" loads have been under-estimated. Hence the Meyerhof theory would appear to err on the conservative side.

After an extensive review of data for London Clay, Skempton (1959) concluded that an average X value of 0.45 was satisfactory providing an upper limit of 2000 lbs./sq.ft. was imposed on the value of Ca. For badly fissured clay near the surface X might drop to 0.3, but with deeper piles for heavy foundations X might rise to 0.6 under favourable conditions.

The ultimate load may be defined as that load which is reached when the pile first continues to settle at a steady rate under constant load. A new method of pile testing introduced by Whitaker (1961) in which the test is conducted at a constant rate of strain, gives in many cases a definite peak on a load settlement curve.

Time Effects.

The majority of results on bored piles in London Clay indicate that there is little or no significant gain in carrying capacity when tested six months and eighteen months after the first test loading at one month after installation. Thixotropic regain of strength in the clay usually takes place during the first month. In addition unbalanced pore pressures have usually dissipated as is the case for driven piles.

Settlements.

In an article on the long-term loading of short bored piles Green (1961) loaded piles for four years and showed that settlement continued throughout the period of the test, but the rate of settlement was small after the first three months. In these tests the design loads were applied, and 60-70% of the total settlement occurred within the first three months, and 50-60% within the first one month.

Skempton (1959) also studied settlements, but for piles loaded to the ultimate load he observed that for piles between 12-24 in. dia. the settlement was approximately equal to 1 in. per foot diameter or 0.085B where B is the pile diameter. The length of the pile was thought to have little if any influence. At 90% of the ultimate, the settlement averaged about 0.04B. From the results of some tests by Tomlinson, Skempton also concluded that the shaft adhesion was fully mobilized after a settlement of about 0.4 in. for 12 and 17 in. dia. piles. For normal piles of these diameters the ultimate load would require settlement of about 1.0 and 1.5 in. respectively. This showed that the shaft adhesion was mobilized at an early stage in the loading and that the settlement at the ultimate load is essentially controlled by the settlement required to mobilize fully the end bearing capacity.

Further work by Tomlinson (1961) verified this conclusion for model piles with enlarged bases, and independent measurement of the base load and shaft load. He showed that the maximum adhesion of the shaft was fully mobilized when the penetration was about 0.5% of the diameter of the shaft. Also the full bearing capacity of the base (as represented by a value of Nc = 9) is not attained until a penetration equal to some 10-15% of the base diameter has been reached. The minimum penetration movement at ultimate bearing capacity was obtained with the longest possible shaft and the smallest base diameter. Thus if settlements are to be kept small and the ratio of actual load on the base to its ultimate bearing capacity may need to be smaller.

Similar results have been observed by other workers Mohan and Jain (1961) and Fleming and Frischmann (1960) on full-size piles. The latter in a test on a bored and under-reamed pile in London Clay with a shaft diameter of 2'6" and a base diameter of 5 ft. found that full mobilization of shaft load occurred at a settlement of 0.7% of the shaft diameter. They recorded a settlement of 11% of the diameter of the base at the maximum load in their pile test. The slope of the load-settlement curve at the maximum test load suggests, however, that the ultimate load had not been attained.

Part of the foundations for the Shell Building, London, consist of concrete cylinders cast insitu on enlarged bases. The ultimate carrying capacity of a single cylinder was taken as the sum of a pressure acting on the enlarged base of nine times the cohesion of the underlying soil and a shear force on the shaft as 0.7 times the cohesion. A factor safety of 3.0 was allowed when determining the permissible carrying capacities.

With reference to practical site investigation, difficulties are often involved in obtaining satisfactory cores (for unconfined and triaxial compression tests) in marls and partially cemented sandstones of Triassic Age, which exist under a number of large North Western and Midland cities. Together with the difficulty in applying such shear strengths in formulae basically designed for use in soils, it has been found of practical value to date to take standard penetration tests in such soils and relate these values to formulae derived by Meyerhof.

Birch in his paper 'Engineering Properties of Keuper Marl' given to the Engineering Group Meeting of the Geological Society 1964 (Proc. No.1621) states:

> "Strength measurements from laboratory tests on samples obtained by normal sampling techniques seem to be of limited value; insitu tests probably provide the most reliable estimate of bearing capacity. Calculations based on loading tests carried out on piles founded on hard unweathered marl suggest an ultimate base resistance of the order of 45 tons/sq.ft."

Birch does not state whether he is proposing use of S.P.T.'s or Menard Pressuremeter.

Palmer (Symposium on large diameter piles 1961) states:

> "The author's experience is that the relative hardness of soft rocks such as Keuper Marl and Chalk can be assessed by means of the standard penetration test."

Meyerhof's formulae permit easy use of the standard penetration test and in view of the aims of this thesis, it is thought wise to discuss briefly what is involved and how the formulae have worked out in the author's experience, albeit somewhat limited. Again however it should be borne in mind that Meyerhof's formulae are based on field tests in soils made in conjunction with Frankipile Limited.

<u>For Footings</u> Meyerhof suggests:

(1) Using Factor of Safety = 3

Safe bearing capacity = $Qs = \frac{NB}{30} \left(\frac{1+D}{B}\right)$ tons/sq.ft.

where N = No. of blows in S.P.T.

D = depth of footing below ground

B = width of footing

he suggests that for silty sands Qs should be reduced by up to $\frac{1}{2}$. For sand and gravel mixtures Qs can be increased up to 2x depending on higher ϕ values which should be determined by separate tests.

Full submergence of cohesionless soils reduces the effective unit weight and thus the bearing capacities by about one half. Bearing capacity is not affected by a water table at a depth greater than about 1.5B below base level.

Allowable bearing capacity QA may be less than Qs if latter would give rise to excessive settlement. Working on $\frac{3}{4}$ differential settlement and 1" total settlement then

$$QA = \frac{N}{8} \text{ tons/sq.ft.} \quad \text{for } B < 4'$$

and $$QA = \frac{N}{12} \left(1 + \frac{1}{B}\right)^2 \text{ tons/sq.ft. for } B > 4'$$

$$\simeq \frac{N}{10} \text{ irrespective of B.}$$

This means that unless D is large QA will be $<$ Qs if B $>$ 3'-4'.

For rafts and piers he considers that one can use 2x QA (desired for footings).

For Piles Meyerhof suggests:

$$\text{Ultimate bearing capacity } QF = \underset{\text{(end bearing)}}{qp \quad Ap} + \underset{\text{(skin friction)}}{fs \quad As}$$

when Ap = cross sectional area of pile base

As = surface area of pile shaft

fs = average unit skin friction

qp = unit point or toe resistance

Approximately using S.P.T. $qp = \frac{ND}{2}$ tons/sq.ft.

$fs = \frac{ND}{1000}$ tons/sq.ft.

Generally field conditions are variable and experience is required to assess the average 'N' value to be used in calcs. Greater skin friction may be obtained in piles due to the greater lateral compression of the soil. It has been suggested by certain Engineers that the values given are for saturated sand, and are conservative for dry sand, and that where the penetration ratio $\frac{D}{B} < 10$, point resistance must be reduced to approximately $qp = \frac{4ND}{10B}$ tons/sq.ft. where $\frac{D}{B}$ is small use formulae for footings.

A factor of safety of 3 should be applied to QF.

Certain Engineers suggest that for piles passing through compressible material into cohesionless soils, the safe load $\not> \frac{2}{3}$ point resistance ignoring skin friction.

Care should be taken in assessing bearing capacity of pile groups.

The experience of some Engineers is that Meyerhof's figures, considered in conjunction with loading tests to failure, encourage a belief in his work.

Chapter 4.

PRACTICAL EXAMPLES OF CALCULATIONS OF END BEARING CAPACITIES OF PILES ON ROCK.

The following examples are given to illustrate, in the author's opinion, cases where piles are imposing considerable loading on rock in the form of end bearing.

The formulae used are based on a compromise between Meyerhof's and those used by Piling Contractors to support their designs. Discussions held with Contractors specialising in driven insitu types of piles indicate that they tend to lean towards Meyerhof.

(a) KNARESBOROUGH (ref. Drgs. 1, 2 and 3)

The town of Knaresborough is drained to a point on the North Bank of the River Nidd opposite the sewage disposal works which lie to the south of the river. Prior to 1964 the outfall sewer crossed the river in a 15" diameter inverted syphon. Considerable quantities of stone and grit are discharged by the outfall sewers and these deposit in the syphon and reduce the discharging capacity. Whilst provisions were made to remove grit and stones, it was found impossible in practice to get the pipe completely cleared and the maximum measured discharge was only a third of the theoretical discharge to be allowed for. It was recommended to the Authority that a new pipe and standby pipe be laid over the river on a pipe bridge which would also provide for access across the river.

It was decided to support a steel girder bridge on 4 R.C. piers, having a centre span of 65 feet and spans of 17 feet at each end.

A site investigation was carried out prior to the design of the pier foundations. This consisted of one borehole at the site of each of the central piers A and C shown on drawing No.1. Shell and auger boreholes were made in the upper sands and clays and rotary coring methods in the underlying marl and sandstone. The boreholes indicated that the following succession existed at the site:-

	BH.1.	BH.2.
Alluvium - silty fine sand and silty clay:	8'6" thick	6'6" thick
River Terrace deposits - sands and gravels:	2'9" thick	Nil
Boulder Clay:	16'9" thick	24'6" thick
Marl:	6'3" thick	Nil
Magnesian Limestone	Thickness not proved	

Ground water was met at shallow depth in the boreholes, consistent with river level. However, when the Magnesian Limestone was penetrated, water under an artesian pressure was encountered. The water rose to a level of 107.2 O.D. at borehole No.2 and to 106.4 O.D. at borehole No.1. The direction of flow of water in the aquifer was not definitely ascertained, although it appears that a fall in piezometric head exists from borehole 2 to borehole 1.

Standard penetrometer tests indicated that the non-cohesive soils in borehole No.2 ('N' value 33) were appreciably denser than those in borehole No.1 ('N' value 5). The lower boulder clays at borehole No.2 were tested by S.P.T. at 17'4" and 20'9" b.g.l. giving values of 2 blows for 18", which indicated the extremely soft nature of the clays.

Triaxial compression tests (undrained) gave the following results which indicate that the boulder clay in borehole No.1 is appreciably firmer than that in borehole No.2.

Bore-hole	Depth	M/C %	Dry Density lbs./cu.ft.	App. Cohesion lbs./sq.ft.	Angle of Shearing Resistance
1	14'4" - 15'9"	13.7	120.7	4100	0
1	20'0" - 21'6"	21.5	103.8	1420	0
1	25'0" - 26'2"	18.8	108.5	2020	12
1	30'0" - 31'9"	19.2	108.6	5180	0
2	8'0" - 9'0"	23.9	101.1	1190	0
2	13'3" - 14'3"	25.3	97.1	420	0
2	24'9" - 25'9"	19.4	107.9	720	0
2	29'6" - 30'9"	23.9	99.7	370	0

The engineering properties of the soils from the two sides of the river are appreciably different; whilst the type of soils did not vary as one might imagine to be the case if the course of the river had meandered appreciably in the past.

Marl was encountered 6'3" thick in the borehole No.1 underlain by yellow porous magnesian limestone which contained a major cavity from 34'3" to 34'10" b.g.l. Only a thin band of marl however was encountered in borehole No.2 underlain by yellow sandy fissured limestone and blue limestone with marl bands.

Borehole No.1 unfortunately did not go deep enough to prove the continuity of the blue limestone under the river, therefore also making the proving of a fault impossible. However, from the engineering point of view the salient factor of the rock appeared to be that the limestone was also fissured and contained cavities from 33'6" to 34'6" b.g.l. and numerous small cavities (approximately $\frac{1}{2}$" diameter) between 38'0" to 40'0" b.g.l.

The flow of water in borehole No.1 was stopped by driving a tapered cylindrical plug into the marl but the flow of water from borehole No.2 was not successfully stopped.

The load on each pier, after allowing for dead and live loading plus an overturning movement due to a tree hitting the bridge at times of flood, was approximately 80 tons. A study of the soil conditions indicated that the alluvium was soft and variable in density and would make undesirable foundation materials for the piers, not only because of their low shear strengths and probable high compressibility, but also because they lie at such a level that scouring of the underside of the bases could possibly occur either under conditions of maximum flow or if any alterations were made to the hydraulic gradient of the river in the future. Bases founded on the underlying boulder clay were considered but were not acceptable because of the possibility of differential settlement between piers on the two sides of the river. It should be pointed out that the fall available in the pipe crossing the river was critical to the discharge of the pipe and it was not considered practical to be continually jacking up the pipes to allow for this.

Unfortunately the site investigation contractor did not perform consolidation tests on the samples obtained, so the Engineer designing the pipe bridge was not able to assess the amount of differential settlement. It was considered, however, that even if this information had been available, the criticality of the hydraulics flow calculations was such that any such settlement could not practicably be allowed for. It was therefore decided to found the bridge piers on piles.

The site investigation report recommended that the boulder clay would prove suitable soils in which to found piles with a purely vertical load but that in the case of pier 'C' (borehole No.2) should there be any appreciable overturning movement on the piles the earth pressure against the side of the piles might not be sufficient to withstand the overturning movement.

Discussions I have had with a major piling Contractor suggest that in their experience by jacking between two piles in fill, lateral movement occurred at pressures equivalent to a ground loading of 2 tons/sq.ft., but that all tests made in loose and soft soils have never indicated bending or displacement of the piles at greater than 10 feet below site level.

Having decided on piles, it was considered that in view of the existence of cavities in the limestone, a pile formed by boring and hence placing concrete was unsuitable for the following reasons: firstly, that the artesian upflow of water could scour the green concrete and expose the reinforcement cage, and secondly, that the amount of concrete which could be used would be indeterminate.

Three alternatives were then available: firstly, precast driven piles, secondly, driven insitu piles (plug driven and hence concreted as lining tube removed), and thirdly, cased piles. The first alternative was unsatisfactory due to the requirement of a large piling rig and the expense of the cut-off length in view of the fact that the depth of penetration was uncertain. The second alternative was unsatisfactory due to the requirement of a large piling rig and the same disadvantages as the bored pile.

Because of the anticipated difficulties of access for a large piling rig adjacent to a river bank, it was decided to obtain tenders for piles requiring a large rig and for cased piles (concrete piles with a permanent steel lining) which do not have the same requirement. Tenders indicated that the cost of cased piling was only half that for precast concrete piles, and it was intimated that the additional cost was involved in the provision of a small jetty upon which to move the piling frame.

The final design consisted of two No.14" diameter cased piles under each pier, each pile to be driven outwards at a rate of 15° to vertical and at

8'6" centres at the underside of the pile cap. Using a cased pile which incorporates a flat steel base plate artesian water was precluded from the pile concrete. As the pile cap necessitated the use of a sheet pile cofferdam for excavation, the rake of the pile was determined by internal timbering to the cofferdam. In driving of the pile a dry concrete plug is inserted into the base of the pile and a hammer dropped inside the casing. By this means a jetty was not required, the only requirement being the provision of a crane with a jib of sufficient length so that it could stand above the river bank and drop the hammer inside the casing and hence drive the pile.

As previously mentioned, the artesian water in borehole No.2 was not successfully sealed after boring, and in fact was encountered during the excavation for the pile caps.

Provision was made to pipe the water under the concrete carpet of the pile cap (before pile driving) and this led to a sump hence it was pumped from inside the cofferdam. As the piles and pile cap were designed to stand without vertical support, the possibility of water scouring out under the pile cap did not constitute a hazard in the view of the reinforced concrete designer.

The author was present during the driving of the piles at piers A, B and C, and observed the rate of penetration which he now attempts to relate to the properties of the soils and give an indication of what proportion of load of the pile is being transferred to the Magnesian Limestone at depth.

The following data refers to all 8 no. piles made at the site:

a. B.S.P. helical welded steel casing 14" internal diameter No.10 s.w.g.

b. Type of Hammer B.S.P. 2 tons.

c. Drop 5 ft.

d. Flat plate shoe welded to casing.

e. Concrete plug 1.2.4 mix Sulfacrete.

The driving resistance of the piles was calculated by the following formula derived by the B.S.P. Limited.

$$Ru = W \frac{(3.0 + h) \times 3.6}{S + 0.5}$$

where Ru = driving resistance

W = weight of hammer (tons)

h = height of drop of hammer (ft.)

S = penetration at the set per 10 blows of hammer (inches).

In connection with certain assumptions to be made by the author, attention is drawn to the Civil Engineering Code of Practice No.4 (1954) "Foundations" page 68, where it states:-

> "The fundamental assumption made in all dynamic formulae is that resistance of piles to further penetration under the permanent load has a direct relationship to their resistance to the impact of the hammer at the time of driving. Dynamic formulae may give reasonably accurate results in gravels, coarse sands and similar deposits which on account of their high permeability permit the free movement of their moisture content and therefore do not present a substantially different resistance to the impact forces of driving than to the subsequent permanent load."

Table 4 page 72 of the Code recommends that a 1% deduction be made to bearing capacities of piles at a rate of 1 in 12 when using dynamic formulae. In the following calcs., this allowance is not made as it is considered negligible in view of the assumptions made.

The fact that two no. groups of two piles were driven within 4'3" of

2 no. boreholes with soil tests affords the opportunity to draw comparison between calculated driving resistance using dynamic formulae and calculated bearing capacity using what are now relatively standard calcs. based on the properties of the soils.

Both pairs of piles indicated that the driving resistance of the two piles, made only 8'6" apart, was considerably different, but for the purpose of the calcs. the average number of blows required to drive both piles has been adopted. A theoretical cohesion value of 0.26 was adopted for the purposes of calculating skin friction, and a bearing capacity factor of 9c. The results of these comparisons are indicated in drgs. nos. 2 and 3.

In the cases of piles C_1 and C_2 in the upper softer soils to a depth of approximately 25 ft. the ultimate bearing capacity by 'soils' calcs. is greater than that derived from dynamic formulae by varying percentages but is approximately equal where skin friction only is included (i.e. where end bearing is considered to be negligible, as indicated by low 'N' values).

In piles A_1 and A_2 the theoretical 'soils' calcs. are in excess of the results of dynamic calcs.

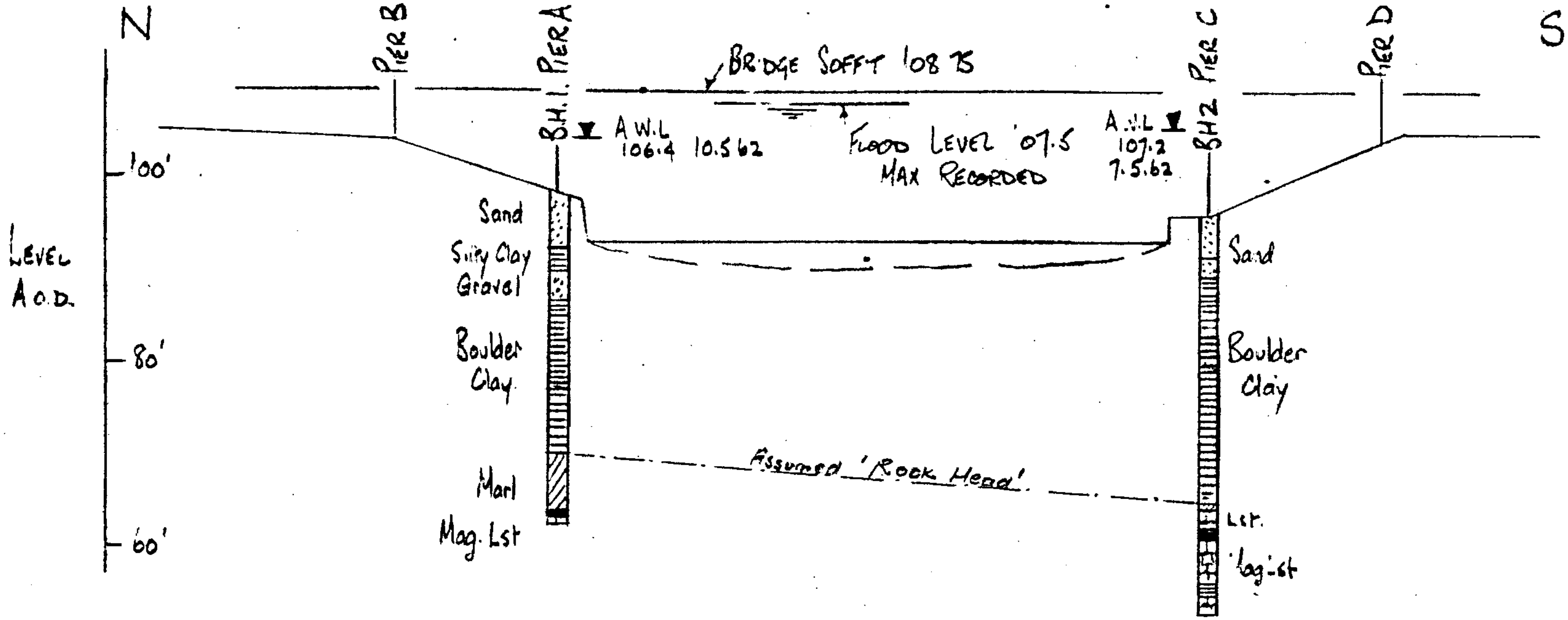
The author feels the results tend to indicate that the skin friction values may be approximately correct but that the end bearing values at horizons above final set are distinctly at variance. It should be stressed, however, that the author's intention in making the comparison was to attempt to analyse what load was being carried by the rock. If one assumes the rock is compressible (however slightly) and that the skin friction derived by theoretical soils calcs. is approximately correct the following safe loads are being carried by the rock:

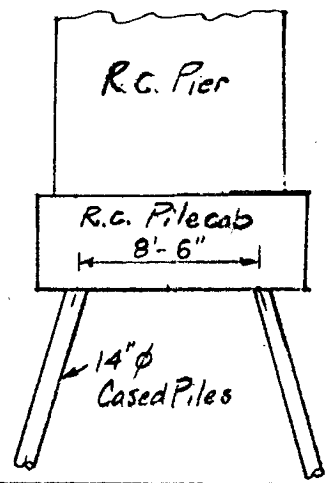
Piles A and A_2	27 tons/sq.ft.	Marl
Piles C and C_2	34 tons/sq.ft.	Limestone.

By comparison the Code of Practice (page 28) gives maximum safe bearing capacities of 10 tons/sq.ft. for clay shales (if marl) and 6 tons/sq.ft. for hard solid chalk (if sandy limestone). As stated, these figures are based solely on the shear strength of the 'rock' irrespective of any settlement that may ensue and have a factor of safety of 2. With deep cylindrical foundations on clay soils the end bearing is calculated by piling contractors as $\frac{9c}{\text{Safety factor usually 2}}$ which means that the safe bearing capacity at depth is approximately 100% greater than in shallow foundations. The author is not aware of any research carried out to ascertain similar values for deep foundations on rock, but some authorities suggest that the end bearing capacities of large diameter bored piles on hard rock is governed by the permissible stress on the concrete of the pile shaft.

Acting in his capacity as Geologist employed by a firm of Consulting Civil Engineers, the author informed his principals of the nature of cavities in the limestone, before piling. The Engineers considered this, and came to the conclusion that as the piles were raking, and also because of the unknown degree of penetration which could be achieved in the rock, if the piles were driven to a final set equal to $2\frac{1}{2}$ x the working load, they would either penetrate cavities of unknown extent or, in the event of refusal, they would be resting on material which would afford sufficient bridging capacity over cavities.

The toes of piles C_1 and C_2 lie about 1'9" above a 12" cavity and those at A_1 and A_2 lie about 1'6" above a 7" cavity. No settlement has been brought to the notice of the author, and one must assume the rocks are satisfactorily carrying their working loads of 40 tons.





Sketch showing Pile Relationship parallel to River

NATURAL SCALE 1 INCH = 20 FT.

KNARESBOROUGH

SECTION ILLUSTRATING SITE CONDITIONS

DRG. NO. 1

Level O.D.	Remarks	S.P.T. 'N' Value	App Cohes. c lbs/sq.ft	Le of Int. Frict. °	Blows for 12" Pen. Pile A_1	Blows for 12" Pen. Pile A_2	Driving Resistance R_u tons	Theoretical End bearing 9c x 1.13 sq ft	Theoretical Skin Friction L x 3.8 sq ft x 0.2c	Total Theoretical Bearing Capacity ULT.
91.58	Top of carpet to pilecap									
89.50	Silty clay								neglect in calcs.	
88.08	Sand + Gravel	33			10	6	3.7	?	say 1000 psft. L = 2.75 0.9	
86.75										
83.66	Stiff brown sdy boulder clay		4100	0	3	5	1.6	18.7	Avc = 4100 L = 3 4.1 + 0.9	23.7
78.00			1420	0	4	5	2.0	6.4	Avc = 2760 L = 8.75 8.2 + 0.9	15.5
77.00										
73.00	Firm red/brown sdy boulder clay.		2020	12	9	9	4.1	9.2 +	Avc = 2395 L = 13.75 11.0 + 0.9	21.1 +
70.00										
68.00	Red marl		5180	0	14	30	9.7	23.5	Avc = 3180 L = 18.75 20.2 + 0.9	44.6
65.53	Toe of Piles A_1 + A_2				S = 0.04"	S = 0.04"	106.6		Avc = 3180 L = 21.2 22.8 + 0.9	
65.00										
63.95	Yellow silty marl									
63.37	CAVITY									
62.50	Limestone.									

END BEARING CAPACITY ON MARL BY THEORETICAL CALC.

= 106.6 − 23.7 = 83 TONS BEARING ON AREA 1.13 SQ FT.

= APPROX. 73 TONS/SQ.FT

PILE DESIGNERS WORKING LOAD = 40 TONS

$\therefore$ SAFETY FACTOR = $\frac{106.6}{40}$ = 2.66

$\therefore$ BY INFERENCE SAFE BEARING CAPACITY OF MARL IS OF ORDER 27 TONS/SQ FT.

KNARESBOROUGH

COMPARISON OF PILE BEARING CAPACITIES - PILES A_1 AND A_2

DRG. NO. 2

Level O.D.	Remarks	S.P.T. 'N' value	App. Cohes. c. lbs/sq.ft.	L^e of Int. Frict ϕ	Blows for 12" penetr. Pile C_1	Pile C_2	Driving Resistance R_u tons	Theoretical End bearing 9c x 1.13 sq.ft.	Theoretical Skin Friction L x 3.8 sqft x c x 0.2	Total Theoretical Brg. Capacity ULT.
91.44	Top of carpet to pilecap								neglect in calcs.	
89.15	Loose sand	5								
87.44	Red/Brown Boulder Clay with soft patches.		1190	0	2	6	1.9	5.4	Av_c = 1190, L = 1.7: 0.6	6.0
84.40			420	0	3	4	1.7	1.9	Av_c = 805, L = 4.7: 1.2	3.1
78.82		1 for 9"			2	4	1.4	negligible	Av_c = 536, L = 10.8: 1.9	1.9
74.90		2 for 18"			½	6	1.7	"	Av_c = 402, L = 14.2: 1.9	1.9
70.90			720	0	6	10	3.7	3.2	Av_c = 404, L = 18.2: 2.8	6.0
66.15			370	0	25	20	10	1.7	Av_c = 364, L = 23: 2.8	4.5
64.65										
63.94	Red marl & sandy Lst. — Toe of Piles $C_1 + C_2$				S = 0.04"	S = 0.04"	106.6	? see below	Av_c = 364, L = 25.2: 3.1	
62.65										
62.15	Cavity — Limestone with cavities.									
61.15										

END BEARING CAPACITY ON LST. BY THEORETICAL CALC.

= 106.6 − 3.1 = 103 TONS BEARING ON AREA 1.13 SQ.FT.

= APPROX. 91½ TONS/SQFT.

PILE DESIGNERS WORKING LOAD = 40 TONS

$\therefore$ SAFETY FACTOR = $\frac{106.6}{40} = 2.66$

$\therefore$ BY INFERENCE SAFE BEARING CAPACITY OF LST. IS OF ORDER **34 TONS/SQFT.**

KNARESBOROUGH

COMPARISON OF PILE BEARING CAPACITIES – PILES C_1 AND C_2

DRG. NO. 3

(b) LIVERPOOL AREA.

(i) Adlington Street (ref. drgs. nos. 4-9 inc.)

It was proposed to construct two no. 18 storey blocks of flats at a site in Adlington Street, some quarter of a mile North of the city centre. The author was associated with the scheme from the site investigation to completion of the piled foundation.

The investigation was initially confined to one block (Block 2) because an investigation had previously been made at Block No.1. At Block No.2 four shell and auger boreholes were made, one at each corner of the proposed point block, together with 2 no. rotary boreholes to prove the nature of the 'rock'. The results summarised on drgs. Nos. 4-7 inc. indicate that the upper clays gave unconfined compressive strengths varying from 1.5 to 4.5 tons/sq.ft., but that the strength varied laterally across the site. The sandstone (Upper Mottled Sandstone: Bunter) was found to be partially cemented but dense insitu. Drilling yielded generally poor core recoveries from 5 to 50%. In view of previous experience in 'sandstones' of this nature the shell and auger boreholes were taken as far as practical into the 'sandstone', and standard penetration tests gave 'N' values of 70+ for penetration ranging from $5\frac{3}{4}$" to $\frac{7}{8}$" generally increasing in density with depth. Unconfined compression strength tests on what cores were produced, gave results varying from 11 to 23 tons/sq.ft.

The results of the investigation at Block No.2 were appreciably different from that previously made at Block No.1 by a different Contractor, so a further three boreholes were made at Block No.1 (1 no. by shell and auger techniques and 2 no. by rotary techniques). The results indicated that the thickness of clays overlying the 'sandstone' was less and that the sandstone, whilst having the same colour and grain size as at Block No.2, yielded higher core recoveries (70 to 95%) and generally higher unconfined compressive strengths (2 to 61 tons/sq. ft.).

Reference to the 6" Geological Map of Liverpool by G.H. Morton F.G.S. 1898, indicated that the site of Block No.1 was intersected by a fault bringing the Upper Mottled Sandstone and Lower Keuper Marl into juxtaposition. A comparison of cores from Blocks 1 and 2 did not however suggest that as far as colour and texture were concerned, any geological fracture existed between the two sites. The difference in unconfined strengths for the two sites may be due to some lateral variation in the deposition of cementing material in the interstitial pore spaces. From the engineering viewpoint the only significance of a fault appeared to be the remote possibility of tectonic movement and the lateral relationship of rocks of differing bearing capacities.

The differing crushing strengths of the rocks is most probably due to scatter which one could anticipate in such partially and sporadically cemented sandstones; the core recoveries may be due to the fact that different machines with different operators were used to core holes on both blocks (although a double core barrel was used in both instances) and the fact that water was used as the lubricant to the bit. The latter explanation would appear to be credited by the fact that the subsequent large diameter augered pile holes proved that at Block 1 the rotary holes did not record partially cemented sand (using open hole techniques) until it reached approximately 7 ft. below that level indicated by the pile holes. However, in the case of Block No.2 the pile holes and rotary holes were comparable in depth.

It is, of course, significant to record that accurate logging is almost wholly in the hands of the foreman driller.

During the course of the design of the piles it became apparent in this type of 'rock' that the creditability and use of the 'N' value obtained by the standard penetration test was greater than that of the unconfined compression test on cores of partially cemented sand, the crushing strength being dependent on the cementing bond between the sand grains.

On the basis of the site investigation it was suggested that the blocks could be built on low-level raft foundations designed to impose $1\frac{1}{2}$ tons/sq.ft. or alternatively on piles taken to the 'rock'. The Engineer concerned decided to use large diameter augered piles and the foundations were designed accordingly. At this stage, however, the City Building Surveyor called for calculations to substantiate the bearing capacity of the piles which varied between 24" and 42" diameter carrying loads of from 77 to 234 tons each.

I investigated the practicability of calculating end bearing capacities of large diameter piles on partially cemented sand, and prepared the following notes:-

"The strata was found to be continuous both in depth and apparent density (as proved by a steady resistance to penetration) but to be of varying degrees of cementation which, because of the irregular repetition in depth of cemented and uncemented layers may not be due to post depositional weathering, but to variations in influx of cementing material.

Theoretical calculations of end bearing capacity on partially and irregularly cemented sandstones appear as yet very limited, and are principally of a practical nature due to the variations in the formation of this type of strata.

Considerable thought has been given to the calculation of end bearing piles in such soils, both the nature of the 'rock' and the effect of lubricating media in any method of core drilling, and the following theoretical calculations have been made bearing in mind the practical aspects of the problem. The calculations are made for guidance only, and should not be taken to form a definite recommendation as to the end bearing capacity of the 'rock' at this site, as the practical experience of the piling contractor (who will guarantee the piles) should be considered.

The standard general bearing capacity formula is as follows:-

$$qf = \frac{1}{2} \gamma BN_\gamma + \gamma D_f (N_q - 1) + {}_c N_c$$

Here it is recommended that for round piles B should be substituted by 0.9 dia.

γ = unit of soil

D_f = depth of foundation

c = apparent cohesion

N_q, N_γ and N_c are bearing factors.

(a) Assume that the 'rock sand' is purely frictional and that no cementation exists between the grains. The term N_γ for frictional soils is low and C = 0 so ignore the terms N_γ and Nc. Curves showing the relationship between 'N' values and ϕ are only approximate, but assume that a frictional soil giving 'N' values of 70+ for less than 12" penetration, has a ϕ value of 40^o. The basic formula neglecting N + Nc can then be written as $qf = \gamma Df \quad (Nq - 1)$

Assume that average length of pile = 35 ft. = Df;

Unit of Soil = 110 lbs./cu.ft. = γ ; and Nq (after Terzaghi) = 90 (but halve to allow for presence of ground water) then substituting:-

$$qf = \frac{110 \times 35}{2240} \quad (45-1)$$

$$= \underline{76 \text{ tons/sq.ft}}.$$

Using safety factor = 3

qa = $\underline{25 \text{ tons/sq.ft}}$. neglecting skin friction.

It has been shown by Meyerhof that for deep slender foundations with a high D/B ratio such as piles, the value of Nq may be + 3 times as great as Terzaghi's value for a shallow foundation.

$\therefore$ say Nq = 300 (for driven piles) but halve this to allow for presence of ground water, substituting:-

$$qf = \frac{110 \times 35}{2240} \quad (150-1)$$

$$= \underline{255 \text{ tons/sq.ft.}}$$

Using safety factor = 3

$$qa = \underline{85 \text{ tons/sq.ft.}} \quad \text{neglecting skin friction,}$$

which is in excess of the compressive strength of normal concrete.

It has also been suggested that the Meyerhof Nq values for driven piles should be halved in the case of bored piles.

An alternative formula suggested is:-

$$qf = K \gamma D \text{ and } N_q$$

where K is a constant which varies from 0.5 to 1

γD is depth of penetration into a granular stratum

A safety factor of 3 should be applied so that $qa = \frac{qf}{3}$

(b) Assume that the bearing strength of the rock is purely cohesive, and that the average unconfined compressive strength = say 20 tons/sq.ft. If the unconfined compressive strength = 2c then c = 10 tons/sq.ft. Substitute in Skempton's formula (remembering however that this was proposed after tests on London Clay)

$$qa = \frac{9 \times 10}{3} + \frac{110 \times 35}{2240}$$

$$= \underline{31.71 \text{ tons/sq.ft.}} \quad ^{*}\text{neglecting skin friction}$$

(c) Using an 'N' value of at least 70 and substituting in Meyerhof p.17

$$qf = qp \; Ap \qquad \text{neglect skin friction}$$

$$qp = \frac{ND}{2}$$

with pile say 35 ft. long.

Safe loading per sq. ft. of pile base = $\frac{70 \times 35}{2 \times \text{Safety Factor } 3}$

= approximately <u>409 tons/sq.ft</u>.

The 'rock' at the site obviously has C - ϕ properties in certain beds and possibly only ϕ values in others, so none of the examples given above is correct in principle. Unfortunately samples obtained could not be tested by triaxial compression methods.

In view of the doubt regarding calculation of the end bearing capacity of piles in 'rock sand' and the opinion that Nq values for driven piles may be two times those for bored piles, it would appear that driven piles have an advantage in end bearing capacity. The resistance of end bearing piles driven insitu would be practically determined on site by dynamic formulae, but should auger piles be used, it is suggested that standard penetration tests performed at the proposed toe of a number of piles could be used practically to check end bearing capacity on site."

The piling contractor submitted the following calculations based on taking skin friction in the clays and end bearing of say approximately 6 tons/sq.ft. on the 'sandstone'. These apparently 'vague' calculations disturbed the Building Surveyor so that he called for a test bore on each site plus an S.P.T. test at the proposed bearing horizon. A study of the calculations indicates that the greater proportion of the load would be taken in skin friction and that the remaining load on the 'rock' would be of the order of 6 tons/sq.ft. Based on his experience, the Building Surveyor said that he would allow 25 tons/sq.ft. end bearing on the sand if an S.P.T. reading of 70 blows for 12" penetration was achieved. At Block 1 the test made at approximately 10' O.D. gave $\frac{7}{8}$" penetration for 100 blows and at Block 2 the test made at approximately 9' O.D. gave $2\frac{5}{8}$" penetration for 100 blows, precautions being taken to prevent whip of the rods in the open pile hole. It should be noted strongly that at these depths (approx. 7 to 10 ft. into 'sandstone') the auger had not met refusal and in fact had not met refusal for an additional 10 ft. of penetration in the test bore at Block 1.

Drgs. Nos. 8 and 9 illustrate the depth of penetration into the 'sandstone' for the completed piles.

PILING CONTRACTOR'S CALCS.

Block 1 - Soil Conditions:-

0' to 11'	Fill and Sand
11' to 30'	Boulder Clay and Sand
30+	Rock Sand and Sandstone.

Consider a 30" dia. pile 45' deep

Friction from 0 to 11' - ignore

Friction from 11 to 30'

S.I. gives cohesion value from 1.3 to 1.9 tons/sq.ft.

Take av. 1.6 tons/sq.ft. with reduction factor of 0.5 and safety factor of 2

$$\therefore \quad \pi \times 2.5 \times 19 \times 1.6 \times \frac{0.5}{2} \qquad = 60 \text{ tons}$$

Friction from 30' to 45'

Assume safe shaft friction of 0.5 tons/sq.ft. in rock sand and sandstone.

Then $\pi \times 2.5 \times 15 \times 0.5 \qquad = \underline{60}$ tons

C/f 120 tons

b/f 120 tons

End Bearing Capacity

Assume 6 tons/sq.ft. on sandstone

$\therefore \frac{\pi}{4} \times 2.5^2 \times 6$ = 30 tons

Total Safe Load = 150 tons

Block 2 - Soil Conditions:-

0' to 3' Fill

3' to 43' Boulder Clay

43' to 50' Rock Sand and Sandstone

Consider a 30" dia. pile 50' deep.

Friction from 0' to 3' - ignore

Friction from 3' to 43'

S.I. gives cohesion value from 0.7 to 2.5 tons/sq.ft.

Take average of $1\frac{1}{2}$ tons/sq.ft.

Reduction Factor 0.5 + Safety Factor of 2

$\therefore \pi \times 2.5 \times 40 \times \frac{0.5}{2} \times 1.5$ = 116 tons

End Bearing Capacity

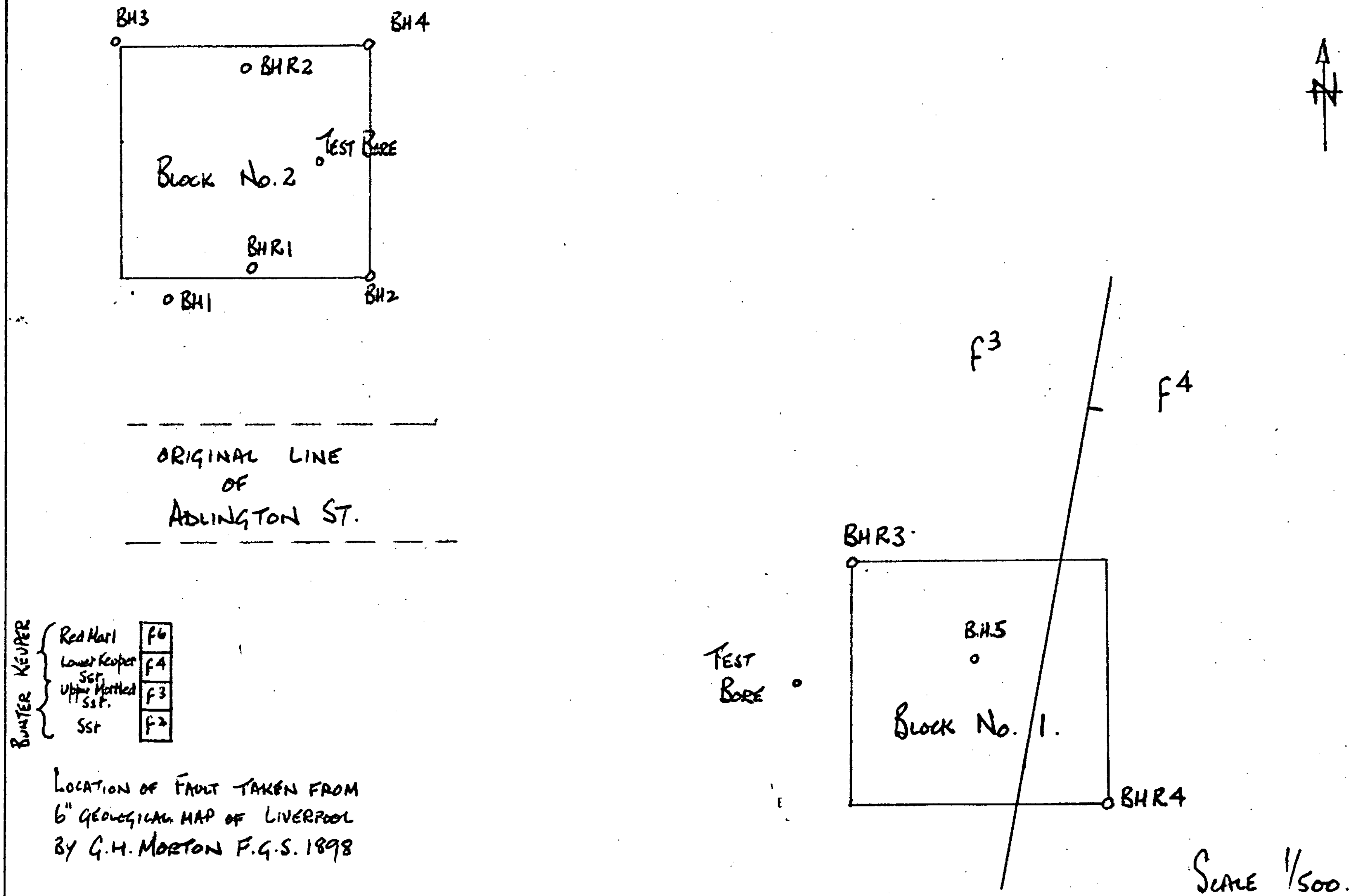
Assume safe end bearing of 6.75 tons/sq.ft.

on the cross sectional area of the pile

penetrating 2 dias. into the rock sand = 34 tons

Total Safe Load = 150 tons

As a result of the S.P.T. tests and having regard to the compact nature of the upper soils, and subject to clearing the pile bottom of loose sand before concreting, the Building Surveyor accepted an end bearing of 25 tons/sq.ft. However, he asked the Piling Contractor to re-submit calculations as he felt that skin friction could not be allowed for piles bearing on rock. During informal discussions the Piling Contractor stated he

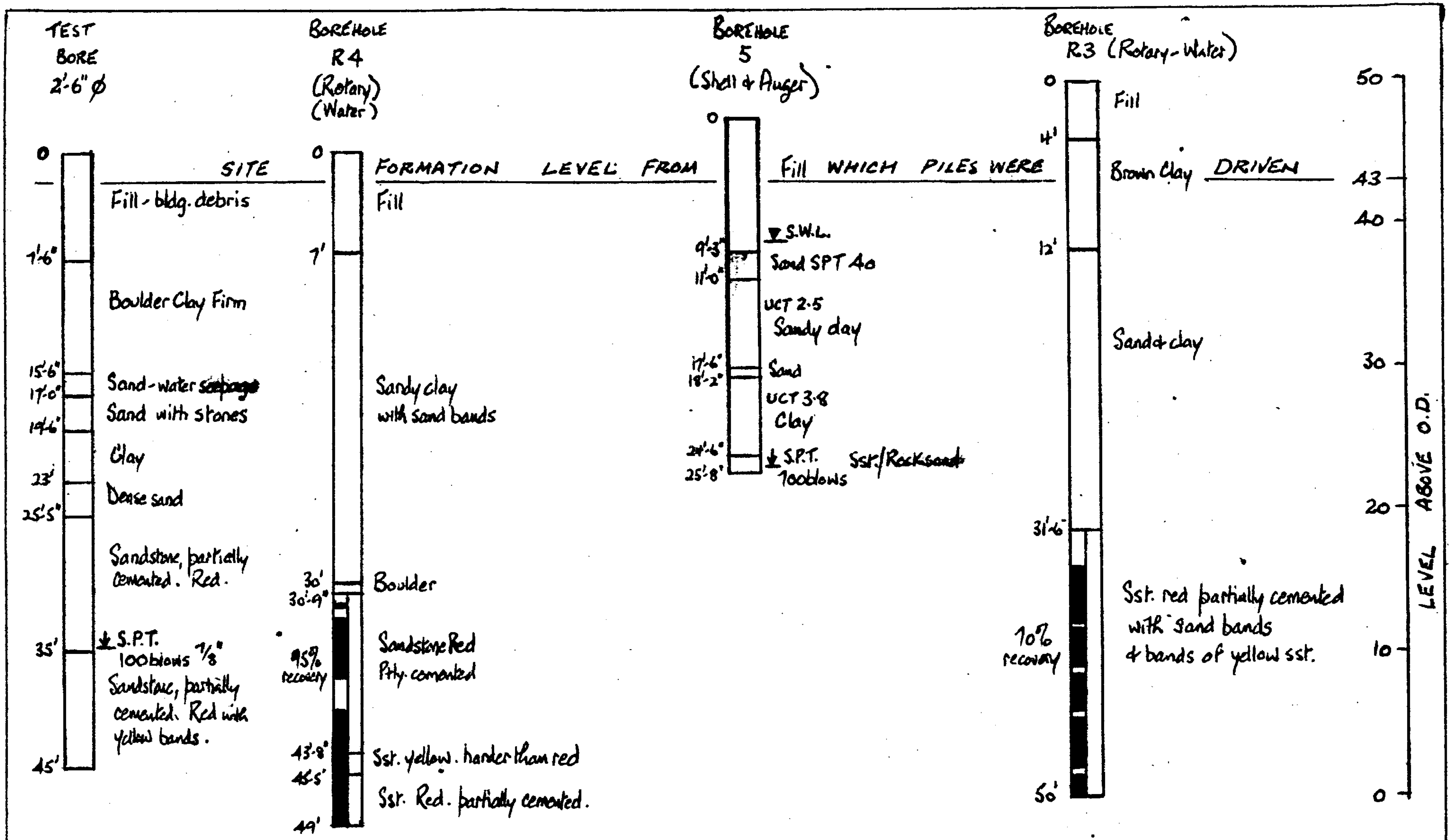


SCALE 1/500.

LIVERPOOL. ADLINGTON ST. REDEVELOPMENT

SITE PLAN

DRG. NO. 4



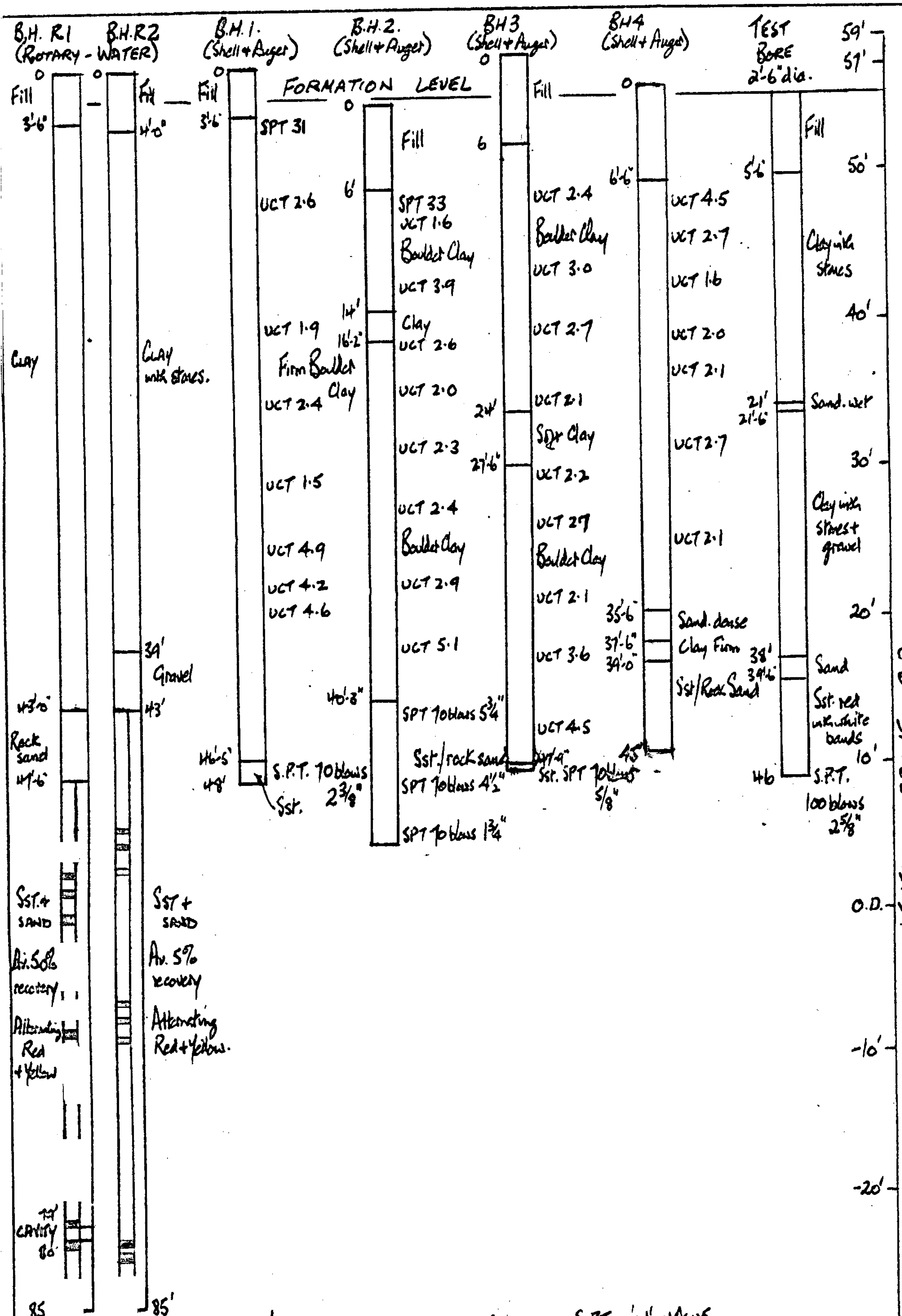
■ Indicates solid core recovery
* remainder of recovery consists of sand.

S.P.T. 40 'N' VALUE.
UCT. UNCONFINED COMPRESSION TEST TONS/SQ. FT.

LIVERPOOL. ADLINGTON ST. REDEVELOPMENT

BOREHOLES. BLOCK ONE

DRG. NO **5**



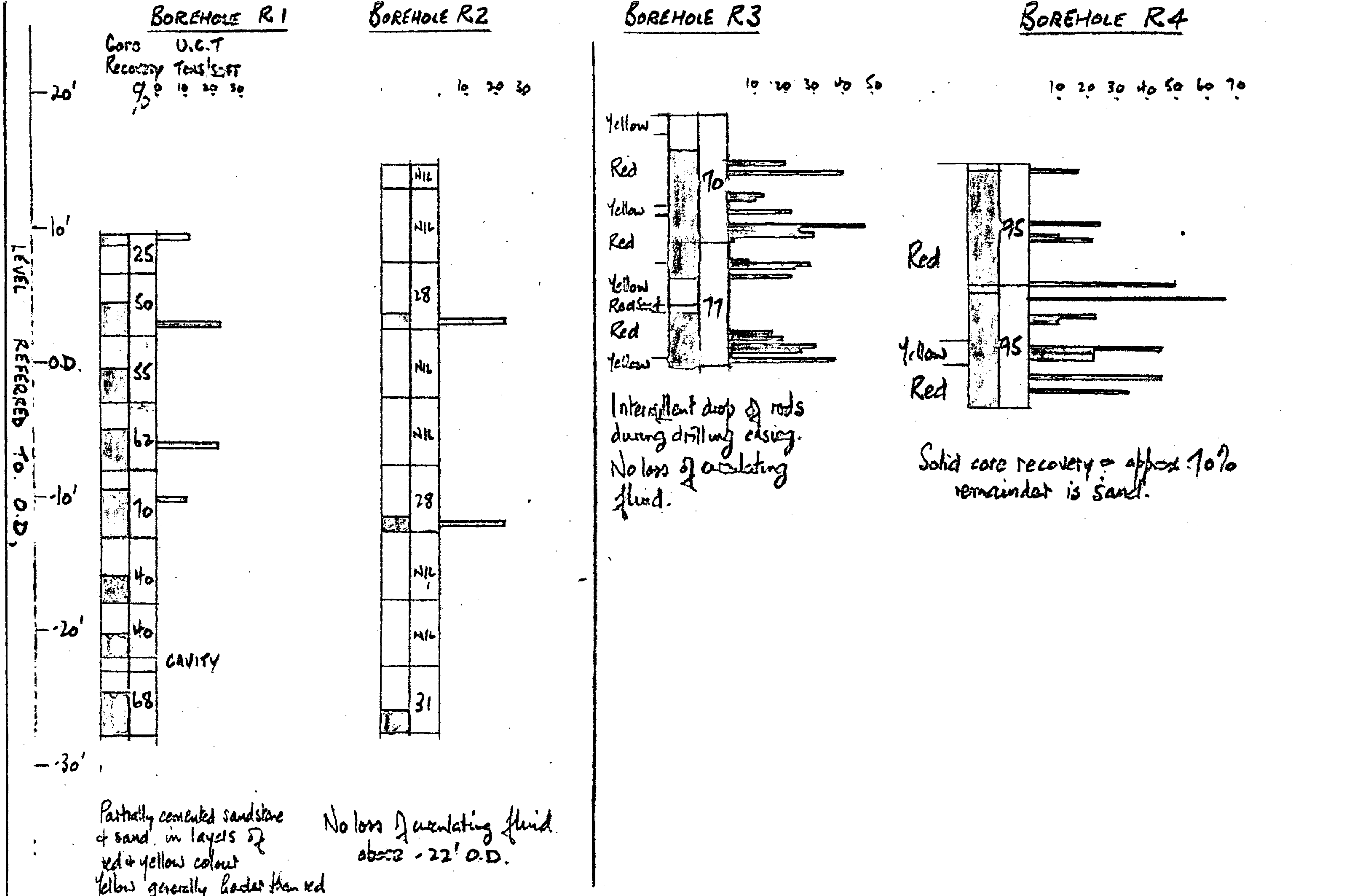
Indicates solid core recovery. S.PT 'N' value.

U.CT. Unconfined compression test tons sq.ft.

Liverpool. Addington St. Redevelopment

Boreholes. Block Two.

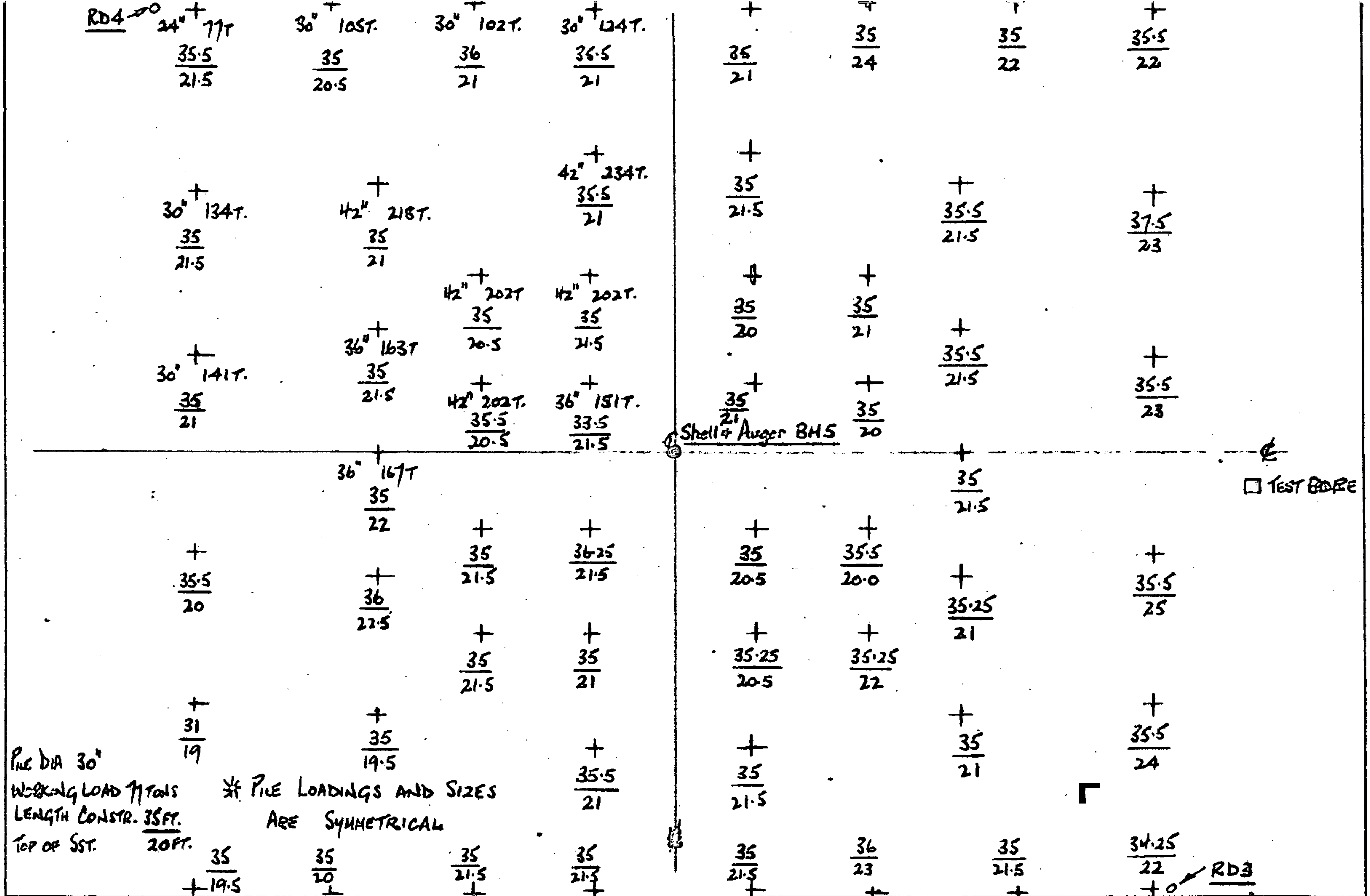
DRG. NO. 6



LIVERPOOL. ADLINGTON ST REDEVELOPMENT

RESULTS OF UNCONFINED COMPRESSION TESTS ON ROCK CORES.

DRG. NO. 7

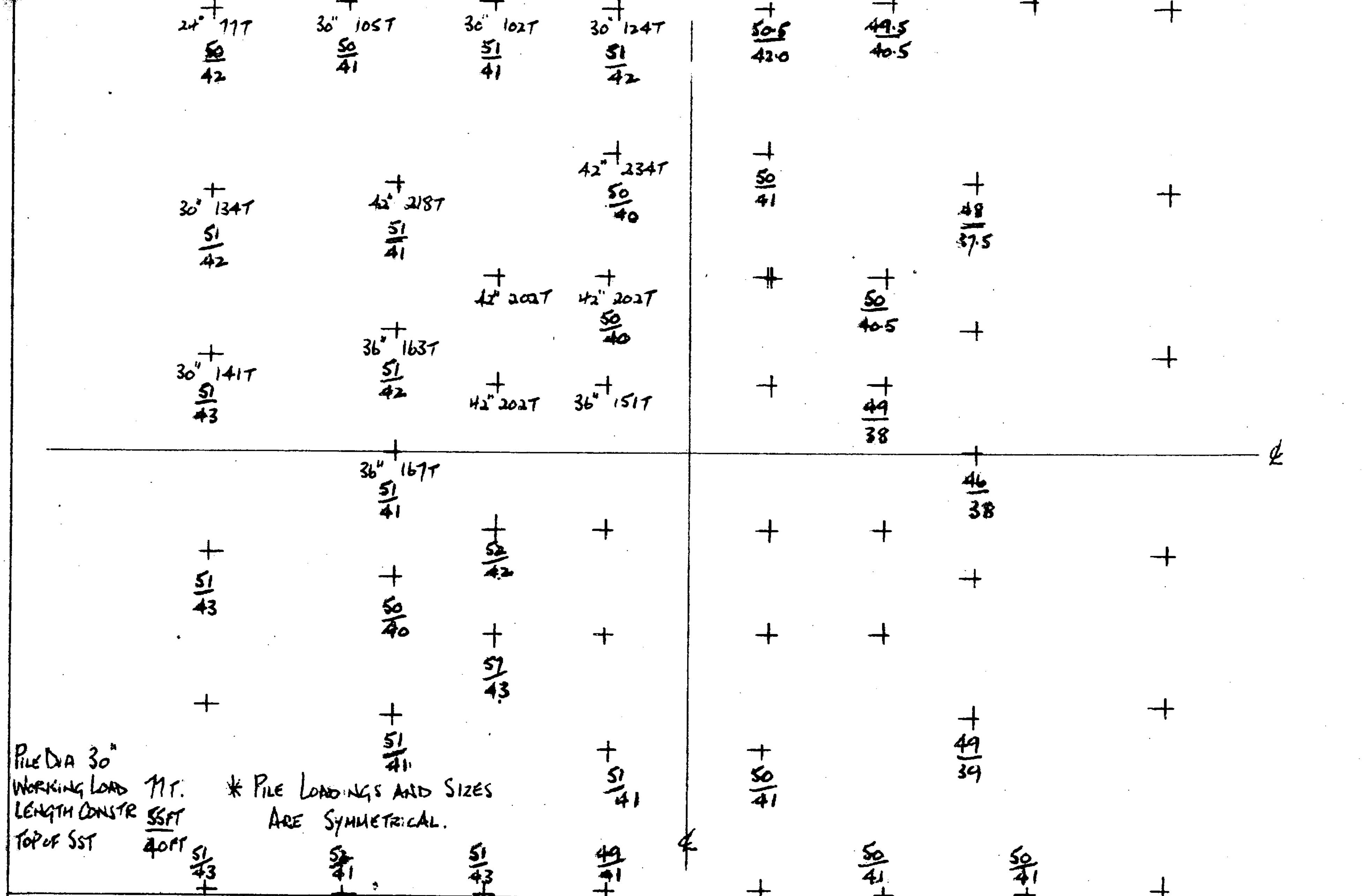


Liverpool. Adlington St. Redevelopment

Block One – Pile Layout

Scale Inchs 10 ft.

DR... NO. 8



Liverpool. Adlington St. Redevelopment

Block Two - Pile Layout

Scale Inch = 10ft

DRG. NO. 9

would be happy in such conditions to design to the shaft concrete strength for piles bearing on rock. Jessop and Eastwood, in their paper in the Civil Engineering Public Works Review, November 1964, entitled 'Effect of Shaft Friction on the carrying capacity of the base of a pile or foundation', suggest that shaft friction in sands has an appreciable effect on end bearing load. Whether these results could be applied is debatable, as they incurred settlement of the base.

Discussions with Piling Contractors suggest that they consider elastic shortening of the pile during application of load will mobilise skin friction which should be taken into consideration. It is evident, however, that difficulty is experienced in the calculation of bearing capacities of piles on such rocks. Obviously some depth factor comparable to Skempton's 9c for clays should be incorporated for a deep cylindrical foundation. Meyerhof's calcs. require the calculation of C - ϕ properties which does not appear altogether practical in these partially cemented rocks due to poor core recovery. Probably some insitu down the hole test such as the Menard Pressuremeter would lend results which one could more readily accept.

(ii) ZANTE STREET (ref. Drgs. Nos. 10-12 inc.)

The following basic information was kindly made available by the Liverpool Director of Housing (Structural Engineer's Department).

A site investigation carried out for two multi-storey blocks of flats was planned as eight holes symmetrically placed at the corners of the blocks - this was not however practical due to access difficulties. Boreholes 1, 2, 4, 8 and 10 encountered mainly competent red and yellow coarse-grained Sandstone with occasional pebbles (possibly the Bunter Pebble Bed). However, boreholes nos. 3, 5, 6, 7 and 9 made in the central area of the site (see site plan) proved disturbed yellow and red fractured Sandstone fragments in a matrix of rock flour and sand.

It was probable that the site was intersected by a fault, with a downthrow of some 500' to the West. Unconfined compression tests gave the following results:-

Borehole number	1	4	4	8
Depth	15'0"	16'0"	24'0"	13'0"
Description	Lt.Br.Sst.	Red. Sst.	Red Sst.	Soft Pink Sst.
Dry Density lbs./cu.ft.	134.0	119.5	119.3	121.0
M.C.%	0.7	7.1	11.3	11.5
Crushing strength tons/sq.ft.	247.5	109.2	60.4	36.2

Unfortunately no cores were obtained in the fractured material but the site investigation report recommended that allowable bearing capacities of from 4 to 5 tons/sq.ft. could be taken for foundations at least 3' wide (this estimate being made by analogy to sand and gravel mixtures) - no S.P.T. tests are recorded as having been made.

The site investigation report recommended that the multi-storey blocks be founded in one of the following ways:-

(a) Excavate to rock head and use space as a basement. Unfortunately the rock head slopes and therefore considerable excavation in rock would be required.

(b) Bored or driven piles taken to rock.

The report also gave approximate limits for the fault zone and suggested that as the eastern block spans the fault there is a risk of differential settlement. It also stated that future tectonic movement along the line of the fault is debatable but suggested that in the case of blocks over 3 to 4 storeys high it would be advisable to relocate the block.

The Engineer considered the above and decided to make a test pile in the fault zone area to check the bearing capacity and therefore obtain an appreciation of what differential settlement could occur. Resiting of the block was carried out in this instance, but principally because this could be easily made without affecting adjacent transport routes.

The results of the test pile are given in Drawing No.12.

The design load on a 20" dia. augered pile taken through 10' of fill, 11'6" of sand, and 8'6" of 'sandstone' was 90 tons. If skin friction was neglected the end bearing would therefore be approximately 42 tons/sq.ft.

The pile was loaded to 2X the working load (i.e. 180 tons) and the final settlement was only 0.09", therefore being well within the $\frac{1}{4}$" settlement usually allowed by Engineers as being acceptable. Unfortunately no S.P.T. results are available in the sand, but the author from his own experience in Liverpool suggests that a safe skin friction of 0.5 tons be taken in the sand and sandstone.

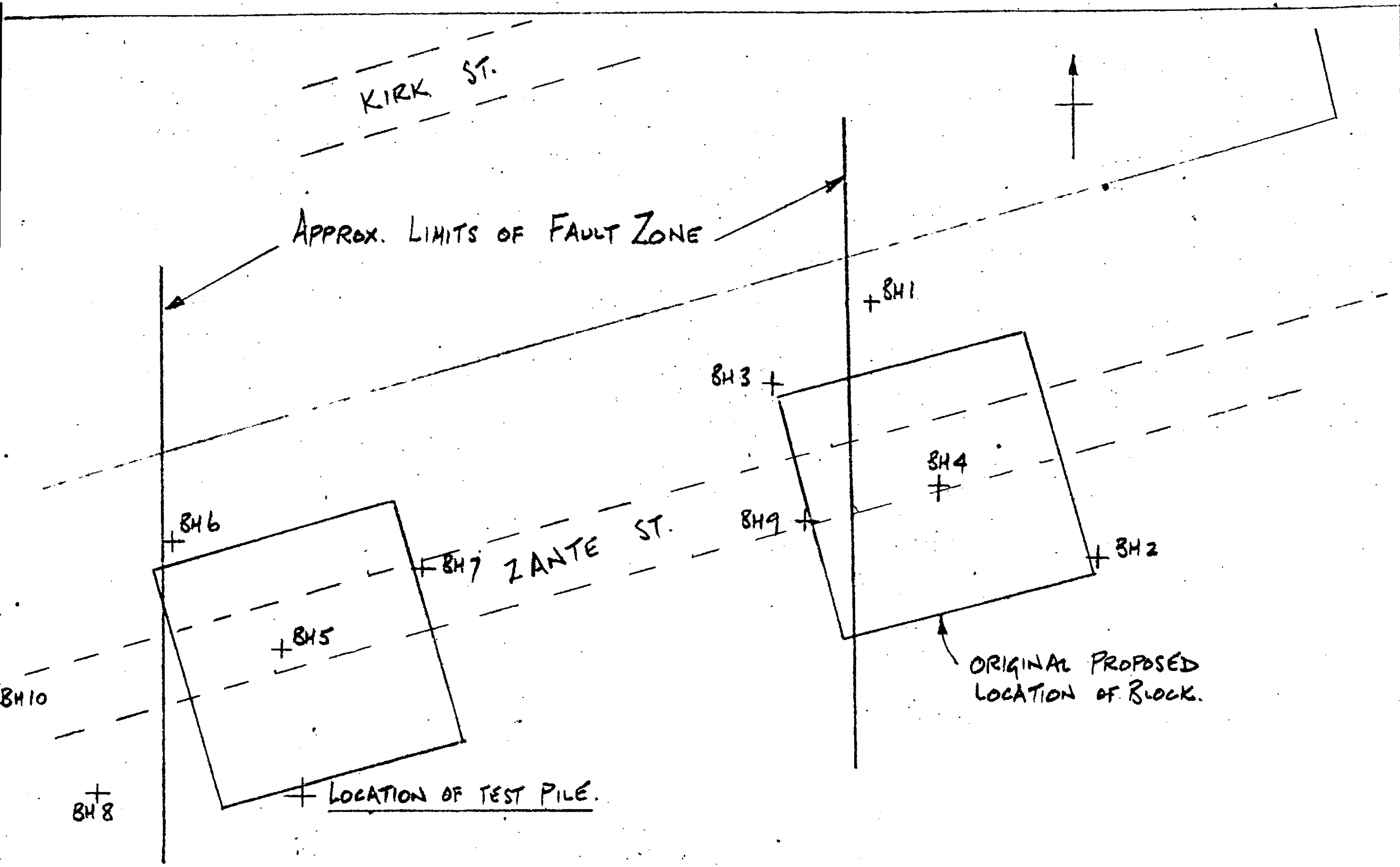
Therefore, he estimates the load being carried by the toe of the pile as follows:-

<u>Skin Friction</u>	0 - 10'	Fill	Neglect
	10 - 30'	Sand and Sandstone	

$$\pi \times 1.66 \times 20 \times \frac{0.5}{2} = 26 \text{ tons}$$

2 —— (Safety Factor)

Assume that with final settlement of 0.09" the pile is capable of a safe bearing capacity of 180 tons



LIVERPOOL. ZANTE ST. REDEVELOPMENT

SITE PLAN

DRG. NO. 10

$\therefore$ End bearing = 180 - 26 = 154 tons

= approximately 71 tons/sq.ft. safe bearing capacity

The author suggests that whilst the above calculation is not entirely adequate, the results do suggest that the end bearing capacity of the pile is in excess of that which we would normally anticipate in rock of the description - sandstone fragments in a matrix of rock flour.

In view of the above, the Engineer had no hesitation in piling the block and in neglecting the effect of differential settlement between piles spanning over fault zone material and 'competent' 'sandstone'.

CONCLUSIONS

The principal causes of inadequacies of foundation design in partially cemented sandstones appear to be two-fold:-

(a) Lack of established theory of the behaviour of such 'soils' under load.

Intrinsically the partially cemented sandstones have C - ϕ properties and therefore bearing calculations should be used which allow for such conditions. Rock sand lies predominantly in Nos. 25 to 200 B.S. Sieves. It is debatable whether skin friction will be engaged and the current feeling is to design only for end bearing although this is disputed in some quarters. The work of Jessop and Eastwood is interesting, but is not conclusive in this respect.

(b) Inadequacies in Testing.

The average depth of the weathered zone (i.e. sand only, without cementing) appears to vary from 5 to 12 ft. thick. Rotary coring techniques yield core recoveries in 'stone' below these depths which are generally inadequate for testing programmes. Ideally the cores should be impregnated and tested triaxially and I am in the process of obtaining economic approval from the City to execute a programme of tests. An alternative would be to use the Menard Pressuremeter in the base of such holes in sandstone.

In the upper weathered zone the standard penetration test has been proved to give reasonably accurate ideas of the density of the sand, although the 'N' values are above any current graphs for interpretation and therefore interpretation has been principally a matter of experience.

It has been suggested that 'N' value (12" penetration) up to 70 are indicative of weathered sandstone and from 70 to 300 + of sandstone.

It has been proved essential (in obtaining good test pile results) adequately to clear the base of the augered pile hole. It is considered the opinion of many Engineers that settlements in excess of ¼" are due primarily to compaction of loose debris in the pile hole, and not to settlement of the natural 'soils'. Test piles, whilst being of interest, do not allow for adequate comparison of formulae and test results, unless they are made to failure. A few Engineers do in fact incorporate such tests on most sites, but unfortunately the numbers doing so at present do not allow for entirely adequate progress in this field.

I understand that one national Piling Contractor who is also a Soils Contractor has made all piling data accessible to one of their senior staff who is engaged in a programme designed to remedy the present deficiency.

Chapter 5.

EFFECT OF DIP AND STRUCTURE.

The effect of dip and geological structures is not always readily appreciated. These considerations are of greatest significance in areas where rock lies near to the surface and where loads are of such magnitude that foundations require to be carried to rock because the soils above are not capable of supporting them.

The two following examples illustrate the significance of the effect of geological structure on foundation design:-

1. HUDDERSFIELD. (Ref. Drgs. Nos. 13 and 14)

When a store extension and development was proposed, the Architect engaged a firm of contractors to conduct a site investigation. Boreholes Nos. 1, 2, 3 and 4 (see site plan and section) were made, although no mention is made as to whether these were shell and auger or rotary, or a combination of both. All four holes proved fill and compact clay with stones to depths of from 7'6" to 9'6" in boreholes 1 and 4, and from 1'6" to 2'3" in boreholes 2 and 3, which were made from basement level (as shown in section). All holes proved weathered coarse sandstone of a thickness from 1'0" to 1'9", underlain by coarse sandstone. Unfortunately the holes only proved sandstone to depths from 8' to 12' and to a thickness varying from 1'6" to 6'0" with a lowest level of 70' A.D.

The report indicated that a high rock level existed over the site with the 'rock head' generally following the ground contour. It was recommended that all foundations be taken to the rock formation and designed to impose a load of 10 tons/sq.ft.

On the basis of the above the working drawings were prepared, contract let, and work on site commenced. Excavations commenced in the new basement area where the formation level was approximately 12 ft. below the penetration of the deepest borehole. At about 2 ft. above formation level the excavation went into black fissile shale with a bearing capacity of apparently less than the 10 tons/sq.ft. for which the foundations had been designed. It should be noted in this instance that 20% increments for 1 ft. increase of penetration into rock did not satisfy the Engineer.

Acting on the instructions of the Clients, I visited the site and instructed the Contractor to make rotary cored boreholes at sites to be determined as results became available. The object of the holes was to prove the continuity of the strata and to obtain cores for crushing. Initially two holes Nos. C1 and C2 were made in the bottom of the excavation for the basement. Both these holes proved loosely bedded fissile black shale to 15 ft. below formation level with core recoveries of 95% average. At this stage it was felt that a further hole C3 should be taken to at least A.D. 27' to investigate the possibility of a coal seam lying below the shale, and also to provide cores of the overlying sandstone. The borehole proved weathered sandstone with joints filled with soft clay to 14'3" (63' A.D.) and black fissile shale to 47' (30' A.D.) and sandy mudstone to 50' (27' A.D.) with core recoveries of av. 95% in the sandstone to 99% in the shales.

A visit was made to the N.C.B. (Yorks. No.6 Area at Barnsley), who said that although the site lies in the outcrop of the Lower Coal Measures, no records exist of any workings having been carried out in the past and no future workings are envisaged. Two seams, the Hard Bed and the Hard Bed Band, outcrop some distance South of the site, but the N.C.B. had no records of thickness and depth. It appeared that no survey of the area has been carried out by the N.C.B., as they were unable to furnish further information regarding the possibility of other productive seams under the site area. The hole C3 was made to explore the possibility of 'strike workings'. The cores were trimmed and crushed (unconfined) to give results varying from 39 to 260 tons/sq.ft. in the sandstone and from 13 to 39 in the shales. The sandstone and shale showed a scatter of results probably due to inclusions of soft clay in horizontal and vertical joints. The shales were extremely fissile and the cores had to be contained in tape before being crushed. After allowing for jointing, bedding and weathering, it was recommended that the basement be founded on shale and designed to impose a load of 4 tons/sq.ft., and the bases at higher level where a thickness of approximately 14 ft. of sandstone overlies the shale designed to impose 10 tons/sq.ft. It was also noted that care should be exercised to remove all loose rock at formation level which may have been recently weathered and softened and disturbed during excavation, before placing blinding concrete. Vertical joints in weathered sandstone were cleared out and backfilled with grout; blinding concrete was used containing a plasticiser and was of high slump to facilitate percolation into any shale which was loose at surface. The basement fortunately was of sufficient area to allow the foundation to be redesigned as a raft imposing 4 tons/sq.ft. Unfortunately the original investigation did not penetrate to adequate depths, which may be attributable to either lack of suitable equipment used by the specialist Contractor, a tight budget placed by the client; the fact that the specialist Contractor had not been comprehensively briefed by the Architect, or alternatively that the scheme was amended after the investigation.

At the time of the work already described, the half of the store fronting on to New Street was in use, and therefore no investigation could be made. After a period of nearly twelve months the Architect requested another visit to site. The main Contractor had excavated three shafts from the existing basement level at the New Street end of the site. Inspection of the shafts indicated that the shale encountered in the deep basement in phase 1 probably extended under the site rising to the New Street end; it was, however, felt that the level of the shale in trial pit 1 was higher than one would have anticipated considering the apparent dip of the overlying sandstone as encountered in phase 1. As the bearing capacity of the shale was obviously less than that of the overlying sandstone, it was considered necessary to prove the thickness of sandstone over phases 2 and 3 of the site area and to interpret the geological structures which will lie below the proposed foundations and which will therefore have a profound effect on relative bearing capacities and so affect foundation design. Boreholes C4, 5, 7 and 8 were made in a location where an 'L' sectioned retaining wall had been designed to impose a load of 10 tons/sq.ft., whilst boreholes C6 and 9 were made to investigate generally the remainder of the area.

Borehole No. C3 made during the previous investigation in the phase 1 area of the site proved that the base of the sandstone and top of the shale was at a level of 62.75. It should be noted that all levels are referred to an arbitrary datum, (A.D.).

Borehole No. C4 was made at the New Street end of phase 2A and proved clay with boulders overlying sandstone (80.77 to 71.27 A.D.), and shale; the top of the shale being 71.27 A.D.

Borehole No. C5 made at the Victoria Lane end of phase 2A proved clay with boulders and sandstone overlying shale at 69.1 A.D.

Borehole No. C6 made approximately half-way along the boundary of phases 2 and 3 proved sandstone from 80.1 to 68.6 A.D. where shale was encountered.

Borehole No. C7 was made at a point between trial hole No.1 and borehole No. C4 in an attempt to prove the location of the fault which was thought to lie in this area. Drilling proceeded to 13'0" below floor level (77.1 A.D.) without encountering solid sandstone or firm shale which would have been anticipated above this depth by relating trial hole No.1 to borehole No. C4. It is possible that the soft shale with occasional boulders which were encountered represent the 'fault zone'.

Borehole No. C8 made at a point approximately mid-way between boreholes C7 and C4 encountered rock comparable with that found in borehole No. C4. Sandstone was struck at 82.1 A.D. and continued to 70.6 A.D., where shale was encountered.

Borehole No. C9 was made in Albert Yard at approximately mid-way along the elevation of phase 3.

Core recoveries in the sandstone varied from 30 to 100% and in the shale, from 50 to 80%, which was comparable with the rocks as described below. The sandstone insitu is apparently medium to fine grained and includes some thin clay bands along bedding planes. It also contains vertical joints, some of which are clay filled.

The black friable shale encountered in the boreholes (with the exception of borehole No. C7), contains some clay bands but is apparently a competent bearing strata. Where the overlying thickness of sandstone is small (i.e. trial holes Nos. 1 and 2), the shale is weathered, soft and clayey to a depth of approximately 7'8" below floor level; however, this material will be removed in excavations and therefore no tests were made to ascertain bearing capacity.

Representative cores were trimmed and crushed and the results of unconfined compression tests in the sandstone varied from 95 to 268 tons/sq.ft. and from 13 to 21 tons/sq.ft. in the unweathered shale.

A comparison of levels at the top of the shale indicates that a fault and fault zone material exist in the line lying between trial holes Nos. 1, 2 and 3 and boreholes C8, C6 and C9. Assuming that the sandstone encountered on both sides of the fault is the same bed, then the fault has a downthrow of at least 20' from the New Street end of the site to the central area of phases 2 and 3. In view of the possible fault zone (as indicated by borehole No. C7), it is possible that the sandstone encountered on the two sides of the fault is not the same bed and therefore that the downthrow is of greater magnitude than 20'. However, from the engineering viewpoint it was improbable that the magnitude of downthrow is of any significance in an area not worked by coal mining. The significance of the fault, however, was that rocks of differing bearing capacity exist in juxtaposition and therefore differential settlement could occur; also a band of softer incompetent materials may exist in a 'fault gouge zone'.

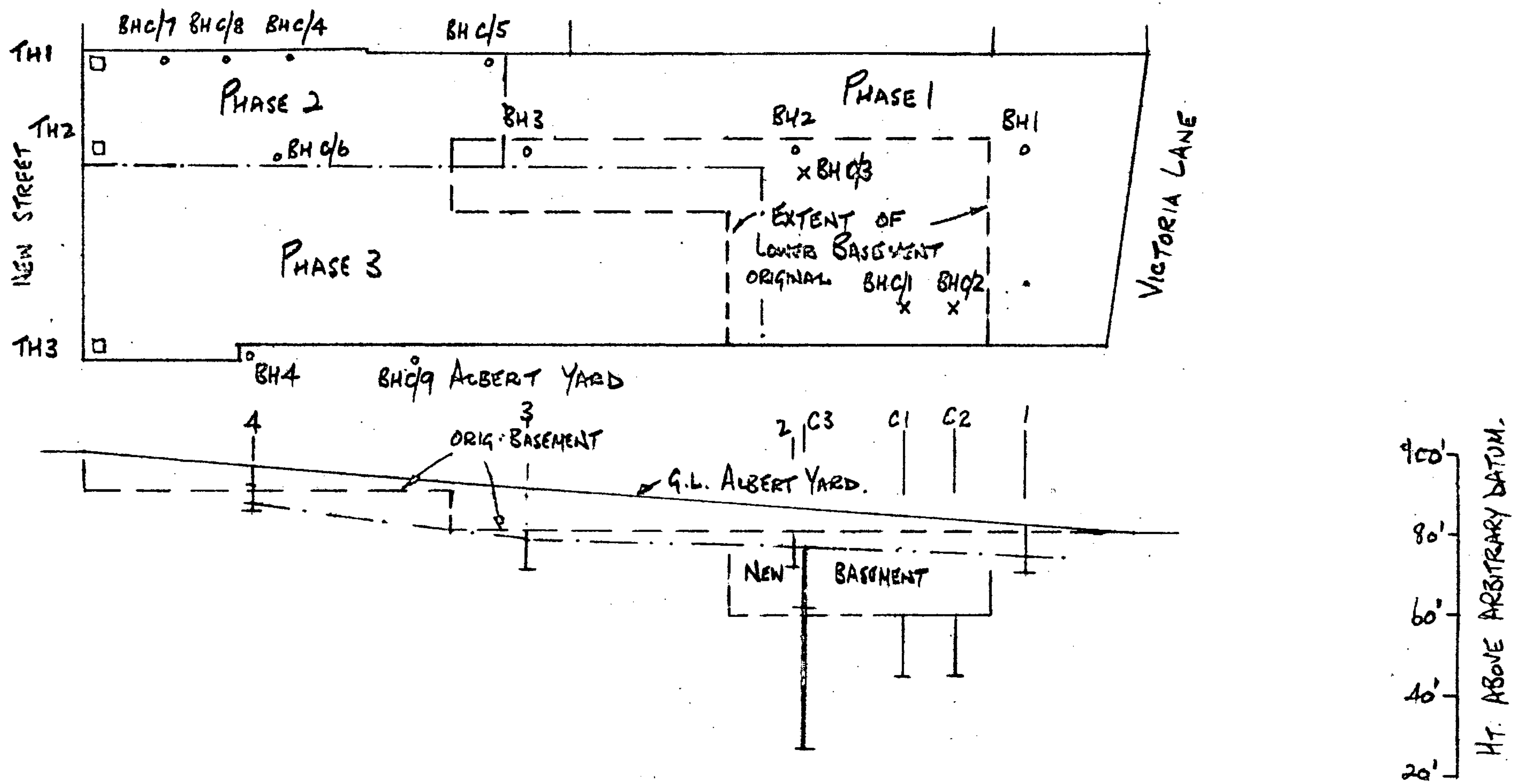
In view of the existence of clay-filled vertical joints in the sandstone and the presence of thin clay partings along the bedding planes, it was recommended that the maximum safe bearing capacity of these rocks be taken as 10 tons/sq.ft. Where the shale is unweathered, I was of the opinion that the safe bearing capacity is approximately 4 tons/sq.ft. However, as indicated in the cross sections, foundations at approximately 79' A.D. will be bearing in certain cases on sandstone where the softer shale is at a variable depth below formation level. In order to reduce to a minimum the effects of differential settlement, it was recommended that careful consideration be given to imposing foundation loads in such a way that the criterion is the safe bearing capacity of the underlying shale.

It is possible that foundations in the area of the 'fault zone' may require additional excavation to reach an adequate bearing horizon. Alternatively, foundations could be designed to span the 'fault zone'.

The use of piles was not considered to be of advantage at the site due to the nature of the rocks and the imponderable precise dimensions of the 'fault zone'. On receipt of the geotechnical report the design engineer altered his working drawings to impose 4 tons/sq.ft. on the shale and 10 tons/sq.ft. on the sandstone where the shale did not lie within less than 5 ft. of the formation level of the foundation. He also designed his footings to span across the fault zone.

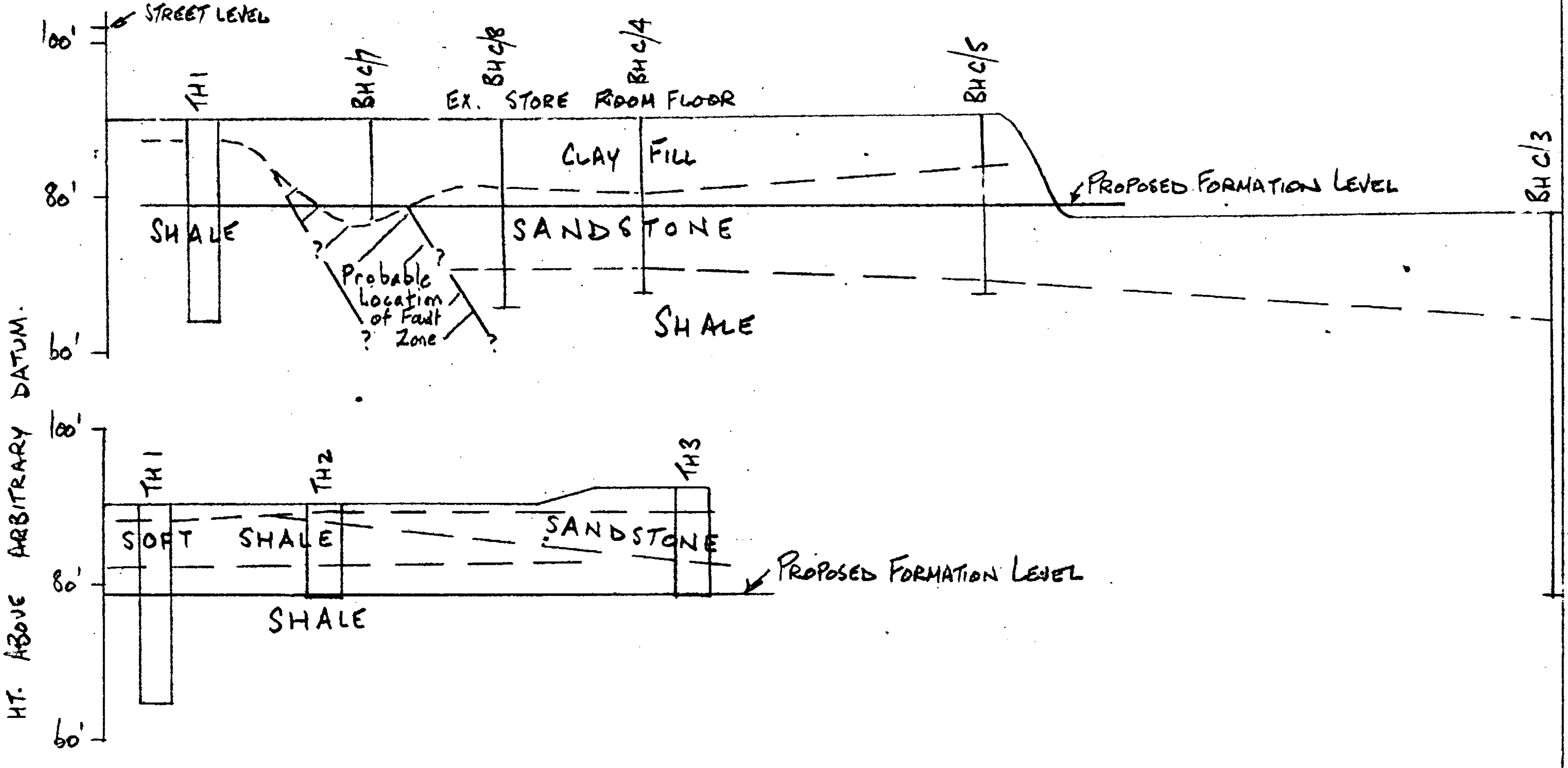
Unfortunately it was impossible to locate the line of the fault due to inaccessible access to areas of the merchandise storage room during the Christmas rush period. The main Client did not consider that any saving in design was compatible with the loss of earnings due to disruption of storage facilities. This in turn meant that the design engineer could not adequately know where to incorporate movement joints into the structure, and this again may involve additional expense.

The author was conscious during the investigation and subsequent discussions with the design engineer, of his own and the Codes' inadequacies



SCALE 1 INCH: 40FT.

HUDDERSFIELD	STORE REDEVELOPMENT SITE PLAN & SECTION	DRG. NO. **13**



SCALE 20FT TO 1INCH NATURAL.

HUDDERSFIELD. STORE REDEVELOPMENT.

CROSS SECTIONS INDICATING INTERPRETATION OF DISTRIBUTION OF STRATA

DRG. NO. 14

in relation to bearing capacities of rocks with clay partings. On occasions such as this one has the feeling of being over-conservative whilst not being able to prove otherwise.

2. <u>GREENOCK, NEIL STREET</u>. (Ref. Drgs. No.15)

The following extracts from the reports written by the author summarise the conditions at the site:-

"The site lies in the bottom of a 'glacial valley' and is adjacent to the Inverkip Road. To the North of the site area the side of the valley rises quite sharply. The Northern side of the valley has been developed previously, but prefabricated houses which once existed have been removed from the area immediately to the North of the proposed site. The site falls from N. to S., approximately 8 ft. across the length of the block.

The proposed development at the site consists of a fifteen-storey point block, to house the tenants of the Greenock Corporation.

According to the Geological Survey sheet, the site is underlain by Mugearite, which is a lava (igneous rock) extruded contemporaneously with the deposition of the calciferous sandstones.

An outcrop of Mugearite was observed immediately to the North of Sutherland Road at a level of approximately 20 to 30 ft. above the general site level.

Several faults are shown to exist in the general site area but none is shown actually crossing the site. It is of interest here to note generally that Geological Survey sheets, whilst giving an interpretation of distribution and structure of rocks, their degree of accuracy is determined by the amount of information available to the Survey and the method of interpretation. Thus one can generally anticipate that in areas of economic workings in geological materials, the information will be fairly accurate, whilst in other areas information is largely gained by observation of outcrops, topographical forms and any random boreholes which may have been made. We suggest in this instance that the data obtained in this report should be made available to the Geological Survey of Scotland.

Rotary drilling equipment was used on your instructions, as it was anticipated that the upper soils were soft and the loads due to the proposed structure being high that piles would offer the best foundation. It was appreciated that with this equipment it is not possible to make tests of engineering significance in the upper soils."

Borehole No.1 encountered peat and red sandy clay with stones to a depth of 13 ft. b.g.l. where a hard boulder was encountered. This was at first thought to be 'rock head', but it was later proved that red sandy clay existed below the boulder, to 16'6" b.g.l. At this latter depth sandstone was encountered. Closed joints were observed in the cores at approximately 10° to the vertical, whilst the 'bedding planes' (containing thin dark shaley inclusions) were inclined at 38 to 45° to the vertical. The sandstone was proved by coring to 25'0" b.g.l. where drilling was discontinued. The core recovery in the upper run was only 60%, probably due to weathering of the upper part of the sandstone. Borehole No.2, made at a lower position nearer to Inverkip Road, proved rock at 25'6" b.g.l. overlain by peat and red sandy clay with small stones. Dark shale with soft partings was proved to 30'0"; core recovery was, however, only 40%, therefore it may be assumed that the partings were soft, as one would anticipate in the weathered top surface of the rock. A thin bed of sandstone 1'7" thick was then proved, but carbonaceous shale was encountered at 31'7" b.g.l. which was proved to be 3'5" thick. At this latter depth (i.e. 35'0") sandstone was encountered which was proved to 44 ft. b.g.l. where drilling was discontinued.

Borehole No.3 proved red clay and boulders to 46'0" b.g.l. where sandstone was found in a 2'6" band underlain by broken black shale and mudstone to 56'0" b.g.l. At this latter depth sandstone was encountered and proved to 64'0" underlain by soft black shale which was proved to 66'0" b.g.l. where drilling was discontinued.

Assuming that the bed of sandstone found in borehole No.1 at 16'6" , in borehole No.2 at 35'0" and in borehole No.3 at 56'0" is the same horizon, then the dip of the strata underlying the site is 1 in 2 in the direction approximately S. 50° W. (approximately 26° from the horizontal). The bedding planes observed in the cores would tend to substantiate this opinion that the rock does in fact dip across the site at a slope of approximately 26° in a South-Westerly direction. Dips of this magnitude are not uncommon in the area. The slope of the 'rock head' is in a similar direction but at a gradient of approximately 1 in 1.6 (approximately 31° to the horizontal).

Samples were trimmed and crushed by unconfined methods, and the results of the tests are shown on the boreholes records. Samples of sandstone gave results varying from 24 to 400 tons/sq.ft. compressive strength. These samples crushed, contained closed joint planes, and failure was observed generally not to follow the line of the joint but to fail by vertical tension cracks.

Samples of shale gave results varying from 32 to 36 tons/sq.ft. compressive strength. It was found difficult to trim these samples accurately due to the presence of joints.

The following recommendations were based on the borehole data, on examination of samples, and the results of site and laboratory tests.

As mentioned previously, the dip of the rocks and the slope of the 'rock head' are inclined in a South-Westerly direction. The postulated geological cross section (along the line of 'true dip') is shown as Section A-B. The limit of the slope of the 'rock head' to the S.W. is not known, but it must be assumed in view of the soft soils above 'rock head' that little lateral stability will be afforded by these soils, and that the 'rock head' may fall more steeply or even form a scarp face in this direction.

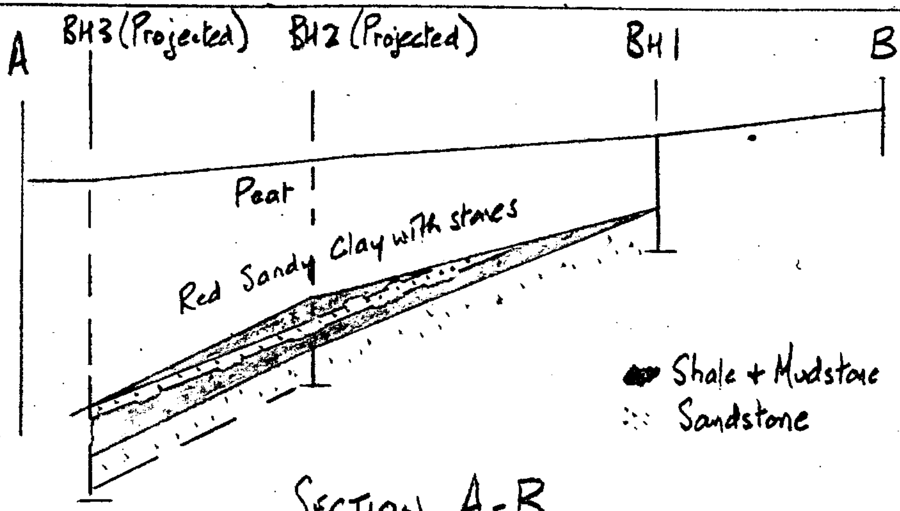
The rock upon which piles could be founded is thought to dip quite steeply across the site. This fact, in conjunction with the interbedded hard and soft rocks (which occur in relatively thin leaves), means that the stability of such piles would be critical. It is also highly probable in view of the saturated nature of the upper soils, the soft condition of the top surface of the rocks, and the dip of the rocks, that ground water is percolating down the surface of the 'rock head', therefore softening the surface of the shales, where they outcrop the underside of the superficial deposits.

In view of the aforementioned problems, the provision of vertical piles only would provide little lateral support to the structure should slip occur along the line of the 'rock head'. We therefore suggest that consideration is given to the use of raking piles on the 'dip' side and 'tension' piles on the rise side. However, further investigation of the line of the 'rock head' on the 'dip' side would be necessary to ensure the practicability of founding such piles due to a sudden fall in the 'rock head', and the acute angle between such piles and the bedding planes of the strata. Because of the thin bedded nature of the leaves of sandstone and the dip of the strata, we recommend that the end bearing capacity of the piles be related to the compressive strength of the shales and <u>not</u> to the sandstones. Piles made over the site will terminate individually in shales and sandstones, depending on the distribution of these rocks below the superficial material. The piles should be 'keyed' into the rock at least 2 ft. The weathered shale will probably be easily penetrated by the pile and it should be ensured that individual piles do not terminate in the weathered strata.

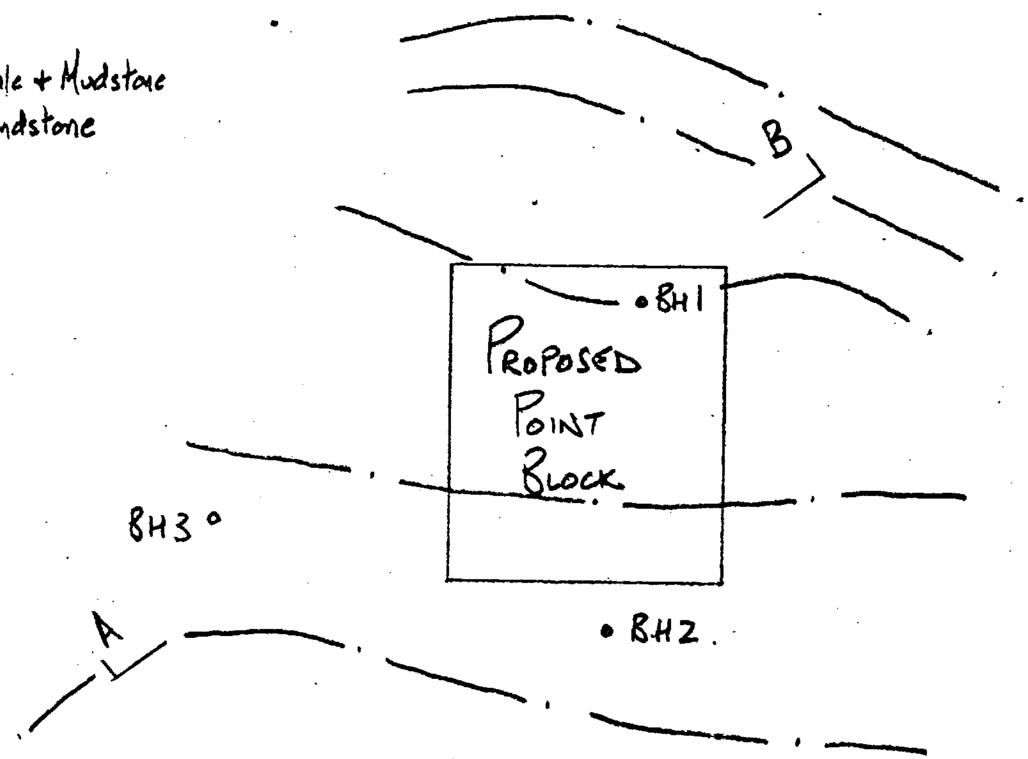
Vertical piles should not exceed an end bearing of 10 tons/sq.ft. The allowance to be made for skin friction is not directly calculable owing to the lack of information regarding the engineering properties of the superficial deposits, nor its lateral stability. Tension piles on the rise side of the Block would only penetrate some 16 ft. through soils of dubious quality, therefore consideration may be given to providing direct anchorage into the rock strata by excavation and design of suitable anchors. The attention of the piling contractor should be drawn to the hazard of terminating a pile on a boulder."

I have since had discussions with several piling contractors who, when asked to design and supply suitable piles, have declined owing to their doubts about the practicability of providing stable foundations at the site.

I understand, however, that work is proceeding on site and that 30" and 36" diameter piles carrying loads from 150 to 200 tons are being used. To obviate dangers due to slip, all piles are being taken to common depth. This of course means an expensive penetration of 20 ft.+ into the rock strata on the 'rise side' of the block. Due to the penetration into the rock, most of the bearing is taken in terms of skin friction in the rock and only a nominal 10 tons/sq.ft. is designed as taken in end bearing on the rock.



Section A-B
Scale 1/500 natural



Greenock. Neil St.
Site Plan & Section

DRG.NO. 15

Chapter 6.

EFFECTS OF WEATHERING.

The effect of weathering of rock is generally a calculable problem in rocks where this 'zone' is sub parallel to the surface of the rock, providing, of course, that foundations are taken to competent rock.

In chalk, however, the effect of weathering has a far greater significance. Testing techniques are somewhat doubtful in their application and design is made with less confidence due to the uncertainty of caverning effects. Chalk, however, varies appreciably in character, as the following examples from Luton and Norwich indicate.

Where limestone and chalk are involved, the prerequisites of good site investigation are the proving of bearing capacity and the location of hidden depressions in the rock head, together with other solution effects in the depth of the rocks.

LUTON. LEAGRAVE ESTATE. (Ref. Drg. No.16)

The site investigation was carried out to assess the best method of providing foundations to multi-storey blocks of flats.

The area lies in the crop of the Lower Chalk which, according to a borehole reported in the Geological Survey Memoirs, is 39 ft. thick in the general site area (the precise location of the borehole is not stated). Underlying the chalk are the Upper Greensand and Gault Clay.

In some boreholes the chalk was found at shallow depth, but in two boreholes chalk was not encountered until 13 to 24 ft. b.g.l. Near the surface the chalk was fairly soft and putty-like, containing small fragments of hard chalk. Relatively hard unweathered chalk was encountered between 30 and 40 ft. b.g.l. with the exception of one borehole, which may have entered a gravel-filled 'pipe'.

Ground water was present at varying levels, as shown on Drg. 16, and no ground water was observed in the hole made in the 'pipe'. Triaxial compression tests were made, but were felt to be unsatisfactory due to the presence of fragments of chalk. Results of three tests ranged between c = 864 to 3168 lbs./sq.ft., and ϕ = 9 to 25 degrees, all samples exhibiting friction. The report suggested that spread foundations near the surface should not be loaded above 1 to 2 tons/sq.ft., and that heavier intensities of loading would result in excessive settlement as the chalk will 'probably' behave in a similar manner to moist clay. The near surface chalk is subject to slurrying with the addition of water, and foundations should be protected immediately after excavation. Any soft pockets present should be dug out to firmer materials. In conclusion, it was suggested that the building be founded on a raft or piles. (Driven piles were in fact used to afford an individual check on the safe working load of each pile as given by the final set. This was felt to be desirable in view of the variable nature of the chalk.)

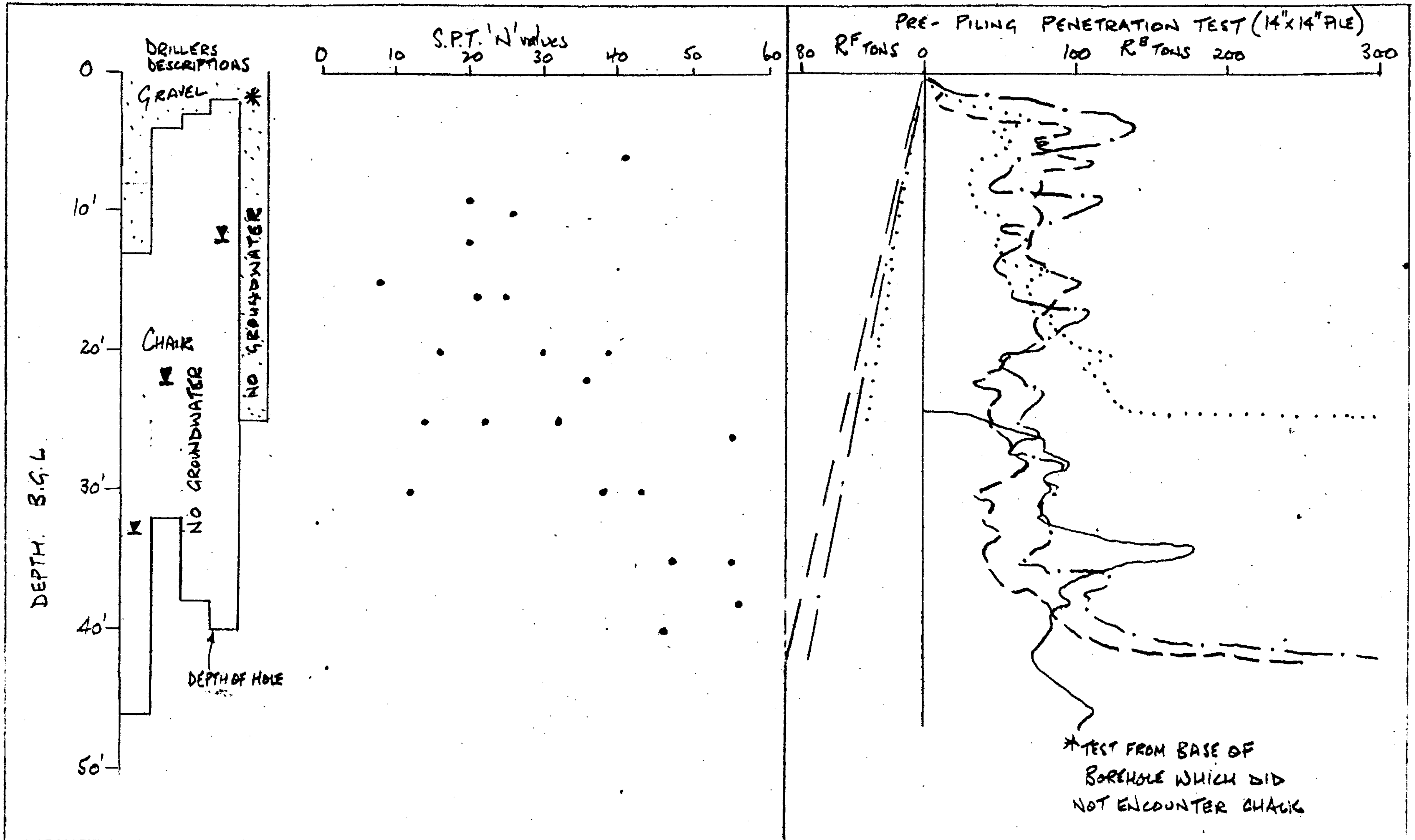
Pre-piling penetration tests were performed to obtain an idea of the length of pile which would be required. In this test a steel probe is driven into the ground by blows from a hammer of known weight and fall. The probe consists of an outer steel tube enclosing a free mandrel, the latter terminating in a point enlarged to the external diameter of the tube. By driving the probe as a whole the total ground resistance is measured, while the point resistance may be deduced by driving the mandrel for a short distance in advance of the tube. The results obtained have been interpreted by the specialist contractor in terms of the driving resistance of a pile of known section (in this case 14" x 14"). The safe bearing pressure for shallow spread foundations is roughly 0.075 of the point resistance R^B. This value is of course subject to settlement considerations, and cannot be applied if a marked reduction in R^B occurs below the level considered. By reference to the test graphs (Drg. 16) it can be seen that at approximately 30 ft. R^B = approximately 90 tons, therefore safe end bearing = approximately 45 tons for a 14" pile (c. 1.35 sq.ft.); at this horizon S.P.T. 'N' values varied from 12 to 43.

However, at depths of approximately 40 ft., where 'N' values are 45+, a high R^B = 300 tons was obtained and R^F = 80 tons. The test is made in such a way that the skin friction R^F is reduced as a separate figure. Therefore as R^B (end resistance) = ultimate bearing capacity, then ultimate bearing at this depth $= \frac{300}{1.35}$ = c. 222 tons; applying a safety factor of $2\frac{1}{2}$

then Safe bearing capacity = <u>approximately 90 tons/sq.ft</u>.
(without reference to depth of loading)

Using the specialist contractor's suggested figures for spread foundations, the chalk at nominal depth would bear 4 tons/sq.ft. The plot of 'N' values at equal depth over the area (500 yards long) indicates generally that the density increases with depth, but that considerable scatter occurs in the 'weathered zone'.

Considerable inadequacies in the testing techniques appear to occur, but one can generally relate within reasonable limits the values obtained by the S.P.T. to the pre-piling penetration test. Triaxial results, however, do not lend the same degree of confidence principally due to scatter and the fact



LUTON. LEAGRAVE ESTATE.
COMPARISON OF TESTS IN CHALK STRATA.

DRG. NO. **16**

that the chalk is obviously anisotropic in character. Pressuremeter tests made insitu would possibly be applicable in soft rocks such as chalk.

The penetration test is based on Hilley's Formula, but has deficiencies, especially in silty soils where a pore pressure resistance can be built up. However, in soft rocks it is considered to give quite a good appreciation of bearing capacity. It should be remembered that the R^F skin friction factor is derived from a 'driven test' and that for bored piles a good factor of safety should be allowed in calculating skin friction.

NORWICH. (Ref. Drg.Nos. 18-24 incl.)

The proposed development consisted of a seven-storey split-level car park with a basement and sub-basement (26 ft. below ground level). The area of the development is approximately 200 ft. x 110 ft. Six boreholes were made originally and indicated that there was approximately 3 to 7 ft. of fill underlain by sand and gravelly sands which extended to depths varying from 25 to 40 ft. b.g.l. where chalk (Upper Chalk) was encountered. This consists of 'putty' chalk which was proved to a max. 65 ft. b.g.l.

The specialist site investigation contractor suggested the use of driven piles designed for a skin friction of 1 ton/sq.ft. However, in view of the deep excavation needed for the double basement it was decided to provide a raft designed for an allowable bearing pressure of 2 tons/sq.ft. at the formation level.

During the last stages of excavations when the last 12" of sand were being removed (i.e. approximately 63' O.D.), some cavities appeared in the base of the excavation at formation level. After closer inspection it was thought that these were formed over shallow holes in the chalk, and the cavity was formed when the overburden collapsed into it. The nature of such cavities is described in the 'Memoirs' of the Geological Survey of the County of Norwich by H.B. Woodward, 1881, as follows:-

> "..... The surface of the chalk is known to be indented or furrowed due to the solvent action of carbonic acid and mechanical abrasion of water. These irregularities are called 'sand galls or pipes' and are filled with deposit that immediately overlies the chalk.
> ..... When the sewage of Norwich was first allowed to flow, we were astonished day to day to find the field covered with circular holes 3 to 5 ft. diameter and of various depths. On one occasion the ground suddenly subsided for a span of 21 ft. and a depth of 12 ft.
> ..... Pipes or sand galls are caused by erosion of chalk and are sometimes suddenly filled by the sand or gravel causing subsidence of the ground; hence the chalk is an uncertain foundation."

With the discovery of cavities it was decided to carry out a full-scale site investigation using shell and auger boreholes, and penetrometer tests. The penetrometer test is an empirical test which gives an indication of relative density and provides a good basis for correlation between tests (see Drg. 24).

These tests made on a 4 ft. grid over that part of the excavation available revealed a large area of loose sand. Boreholes made in this area showed that the chalk was some 10 ft. lower than elsewhere, and that at one position loose sand and cavities extended to 60 ft. below formation level. Additional shell and auger borings were made with four rotary holes around each column centre. At this stage five cavities were disclosed in addition to the soft area (see Drgs. 18, 19 and 20). Remedial work had to be decided quickly. Vibro-replacement and vibro-flotation were considered but discarded because while forming the stone skeletons it disturbs the top layer of the soil. Another possible method considered was the injection of chemical emulsion after casting the raft (using preformed vertical grout holes). However, the cost of this was approximately £10 to £15 per cu. yd. of treated material which was discarded as too expensive.

Finally it was decided to use a rather crude but effective and cheap method of forming aggregate piles; 47 piles were formed in the soft area, in all cavities and under each column. Penetrometer tests made after driving 'piles' indicated that considerable improvement had been achieved; compaction tests showed a 95% compaction.

The work carried out so far was largely practical, but we were satisfied that remedial measures had been effected. We had not considered driving piles into the chalk because S.P.T. tests indicated that the chalk did not increase in strength with increase in depth. The Client, however, decided to obtain a second opinion on the foundation problem affecting this structure.

After the consultant's study of all available information, he called for 13 no. additional boreholes (to 60 ft. depth) together with undisturbed samples which were to be tested by undrained triaxial compression methods and permeability tests. The testing laboratory reported that:

> "To what extent the samples represent the chalk insitu in relation to structure is in my opinion open to speculation, and consequently the value of laboratory tests is questionable."

However, over 250 samples were tested. The permeability tests were never carried out as being unreliable and impracticable. The consultant asked for vane tests, but it was found that the consistency of the chalk strata was such that the normal penetration of vanes was not possible. The distribution in depth of S.P.T. tests, unconfined compression, and triaxial tests is discussed later.

Finally, in view of the inconclusive results of the above tests, the consultants asked for a plate loading test at a given location and depth where a low cohesion value was reported (370 lbs./sq.ft.). During the test an $\frac{1}{8}$" settlement was recorded under a maximum available load of 4 tons/sq.ft. The structural considerations were such that a settlement of $\frac{1}{2}$" under a load of $1\frac{1}{2}$ tons/sq.ft. would have been satisfactory. The test was made at a depth of 13'3" below formation level. Finally the consultants expressed satisfaction and recommended raft foundations based on a uniform pressure of 1 ton/sq.ft.

In view of the number of the tests made at the site, I have attempted to correlate these, and drgs. 21, 22 and 23 indicate these relationships on a site of approximately 200' x 110'. The distribution of 'N' values of S.P.T. tests in depth indicate that the upper weathered zone is fairly regularly soft, whilst with increase in depth the 'N' values indicate that the chalk is of irregular density. Certain high 'N' values were reported by the driller as being suspect due to the presence of flints. Where tests were made in chalk at the base of sand pipes 'N' values fell within the range of those made where chalk was intact from higher level. Unconfined compression tests were made in the putty chalk and generally indicated that with increase in 'N' value, the unconfined compression value increased. The triaxial tests, however, yielded much more variable results. The samples tested were generally creamy coloured, soft, plastic and/or crumbly chalk including fragments of flint. Samples were found to vary in strength in spite of similar moisture content. Individual U4's yielded samples of uniform strength, although moisture contents and densities varied considerably. The laboratory reported that:

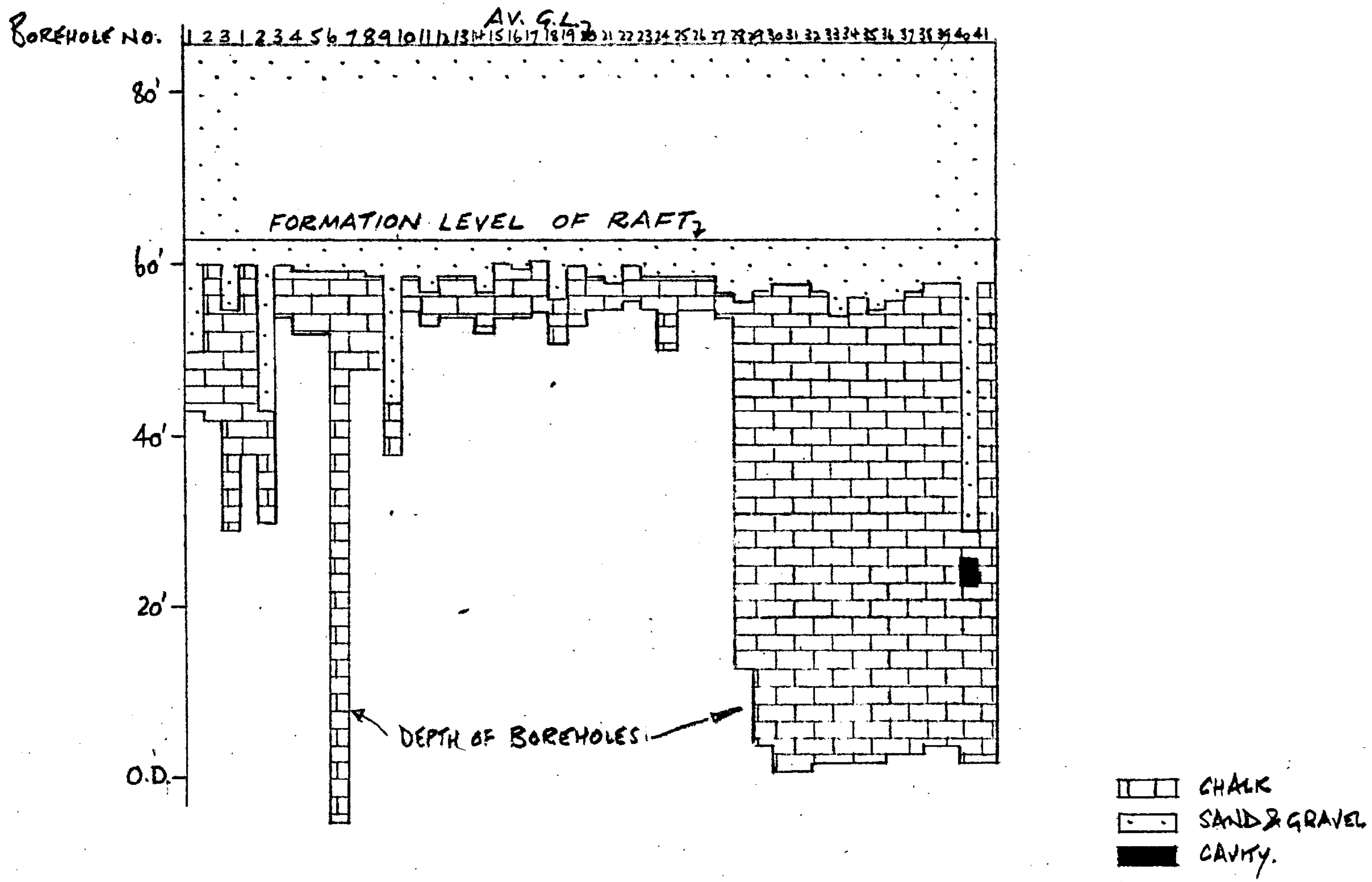
> "The structure of the chalk plays a predominant role in controlling strength properties and that much will depend on the degree to which the material is remoulded in sampling and subsequent handling. To what extent the samples represent the chalk insitu in relation to structure is in my view open to speculation, and consequently the value of laboratory tests is questionable. Individual results were so scattered that interpretation was difficult."

A plot of the relationship between 'N' value and apparent cohesion where friction is zero indicated that for comparable 'N' values cohesion varied between 200 and 2500 lbs/sq.ft. The results of the plate loading test, however, confirmed the general unreliability of the tests.

The results of tests would tend to substantiate the view of Woodward that the chalk is an uncertain foundation. It is probable that insitu tests would be of most practical application in chalk. Perhaps the insitu pressuremeter could be used to advantage as in other soft rocks.

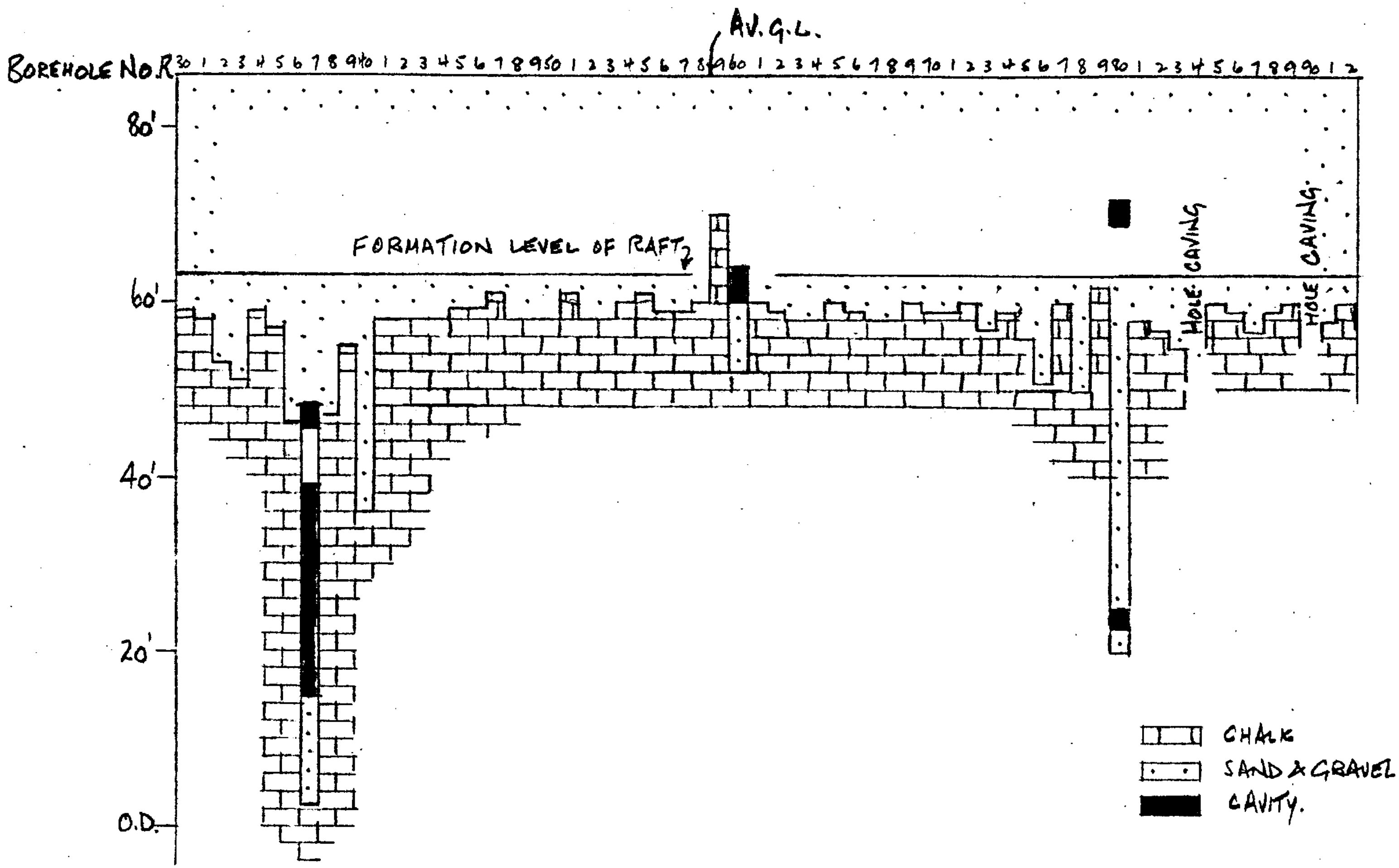
In this instance I agree that piles could not be provided in view of the unpredictable nature of the cavities and that with a raft slab one can design to span over certain areas, providing, of course, that such areas are defined. Whilst numerous cavities, pockets of loose sand and pipes were found and consolidated areas formed, one cannot be absolutely sure that all such areas were located. However, a foundation which spreads the imposed loadings would appear to overcome the difficulties in the most practical manner.

One must not lose sight of the fact that whilst site investigation has been made 'mathematical' by the engineer, there are cases where a practical approach is required of necessity rather than design.



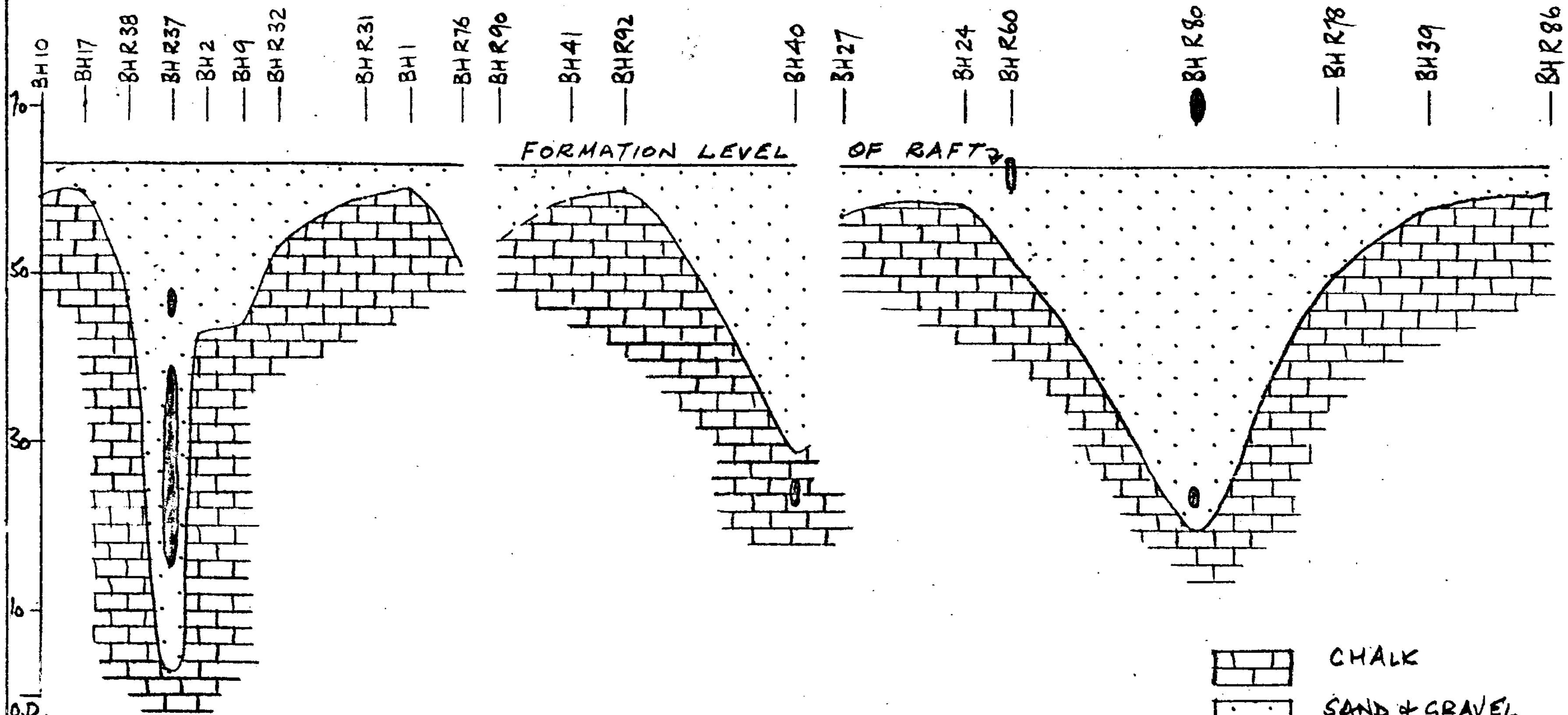
NORWICH. CARPARK
DEPTH OF CHALK IN SHELL & AUGER BOREHOLES.

DRG. NO. 18



NORWICH. CARPARK
DEPTH OF CHALK IN ROTARY BOREHOLES.

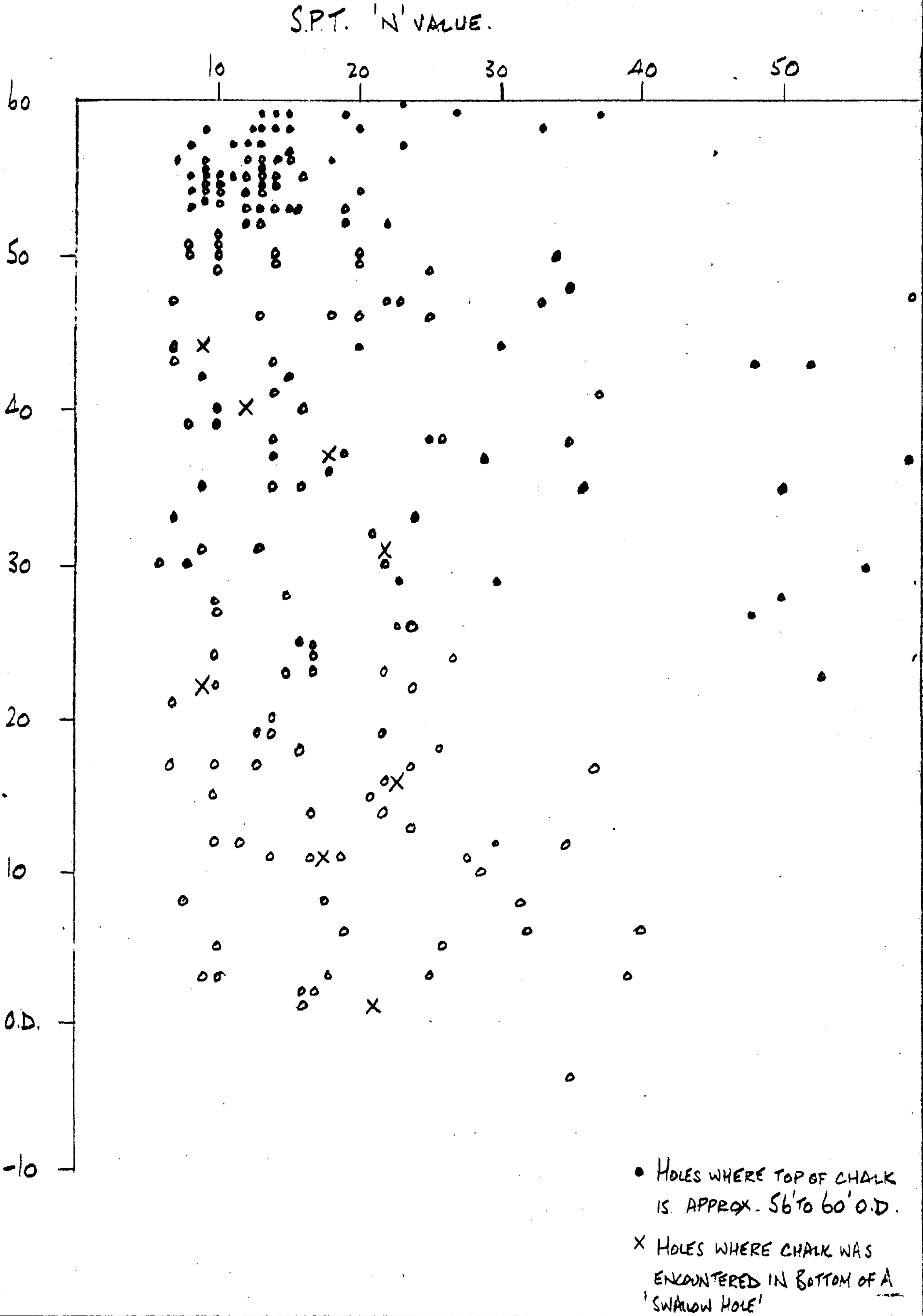
DRG. NO. 19



SCALE 20 FEET TO 1 INCH NATURAL.

NORWICH. CARPARK
TYPICAL CROSS SECTIONS OF 'CAVITIES'

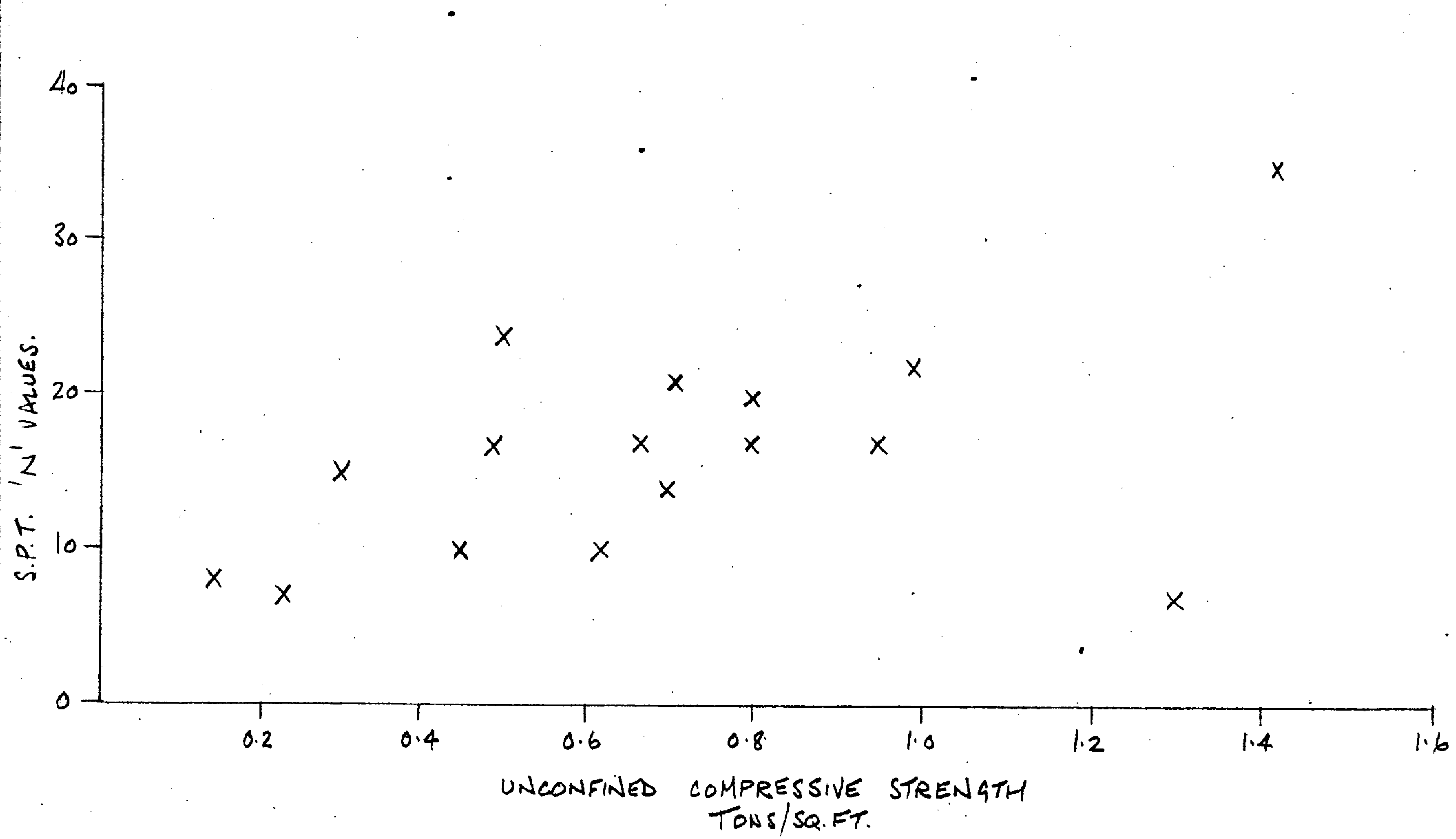
DRG. NO. **20**



NORWICH, CAR PARK

DISTRIBUTION OF S.P.T's IN CHALK WITH DEPTH

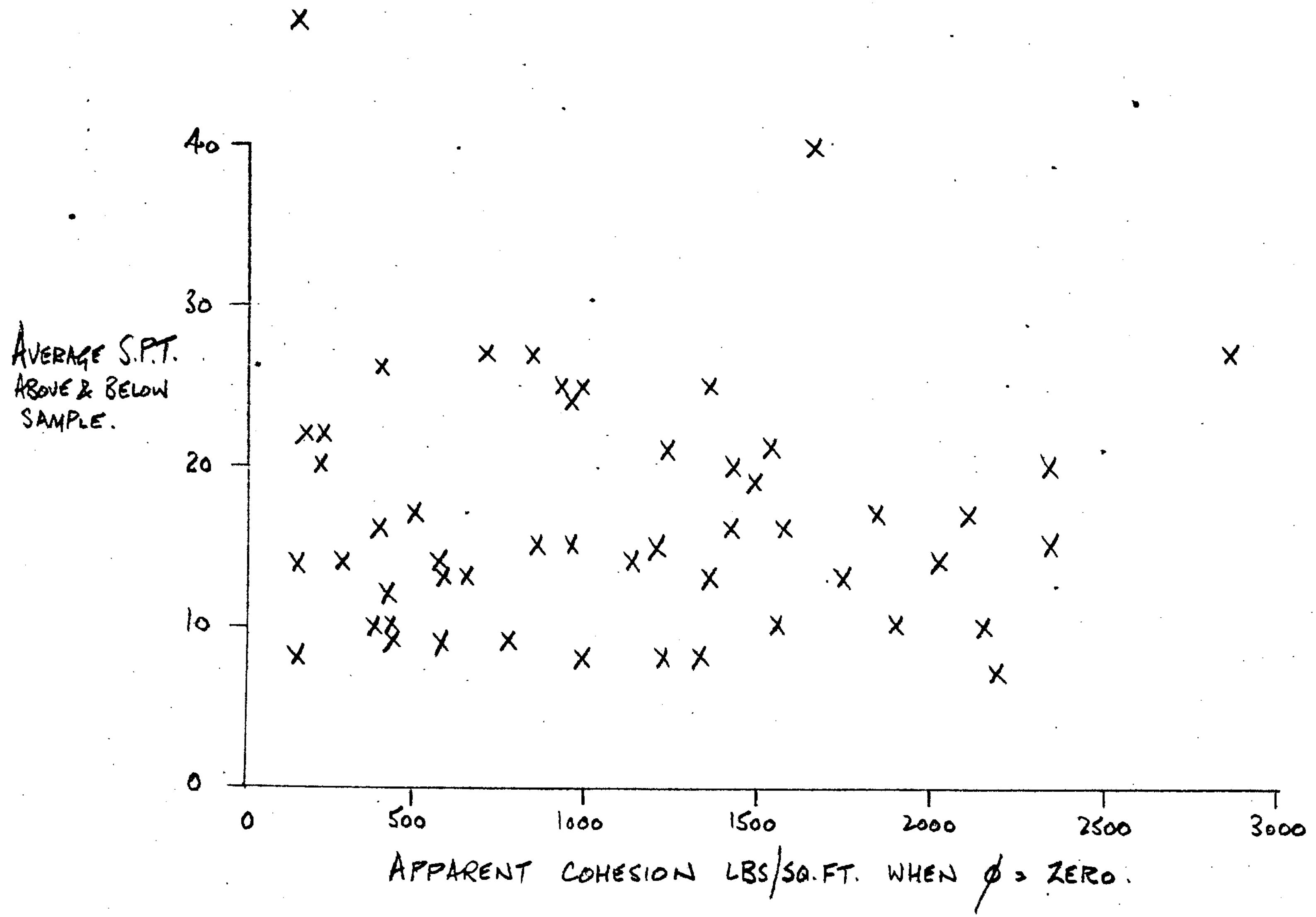
DRG. NO. 21



NORWICH CARPARK

RELATIONSHIP OF S.P.T. 'N' VALUE TO UNCONFINED COMPRESSIVE STRENGTH.

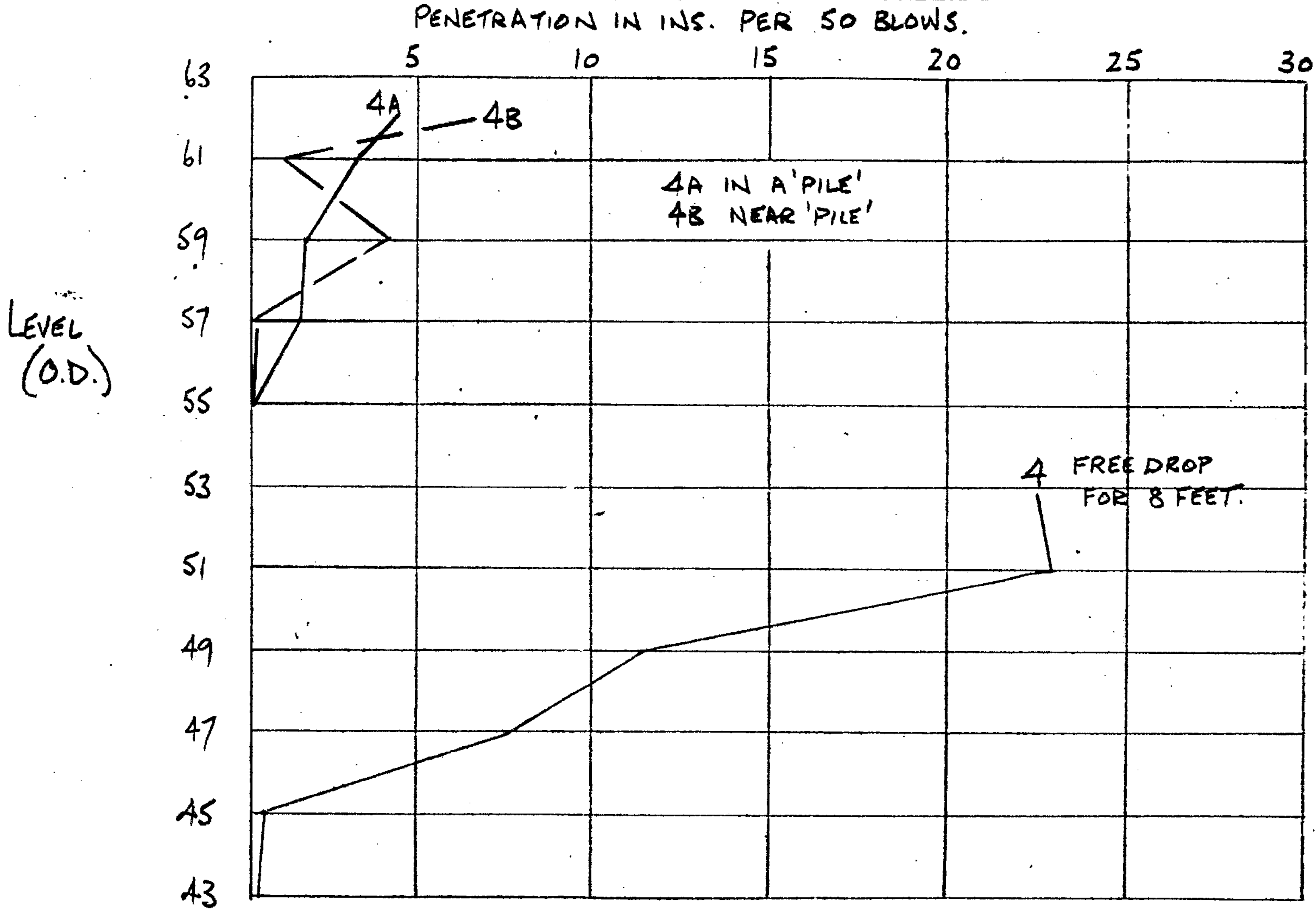
DRG. NO. **22**

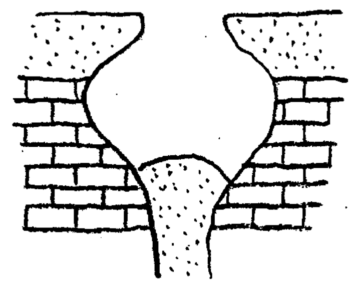


NORWICH. CARPARK.

RELATIONSHIP BETWEEN 'N' VALUE AND APPARENT COHESION.

DRG. NO **23**





Typical Cavity at Formation Level.

NORWICH CARPARK

PENETROMETER TESTS BEFORE & AFTER FORMATION OF 'AGGREGATE' PILES

DRG. NO. **24**

Chapter 7.

EFFECTS OF GLACIATION.

Where heavy loads are envisaged, the effect of jointing combined with glacial erosion poses problems as illustrated by the following example at Bingley, (Ref. Drgs. Nos. 25-27 incl.).

The following are extracts from the site investigation report written by the author in connection with a proposed multi-storey development. I visited the site prior to tendering and formed the opinion that rock lay at shallow depth, and therefore that the principal engineering considerations were the quantities of rock excavation required and the establishment of suitable bearing horizons. The subsequent borings proved that my initial superficial analysis was in error. It was initially decided to make rotary boreholes as the engineering properties of the soils lying above the rock appeared not to be critical.

> "The investigation commenced as described and boreholes 1, 3, 5, 6A and 7 were made. From a preliminary study of these holes it was apparent that a more detailed knowledge of the 'drift' deposits was required. It was agreed with the Architect that the remainder of the holes be made by shell and auger methods to rock and hence by rotary methods.
>
> The site lies on the northern side of the glacial valley in which Bingley is situated. Across the higher part of the site lies the outcrop of the 'Rough Rock' which was encountered and proved in boreholes 1 and 3. Associated with the outcrop are a

number of very large isolated blocks of sandstone which were probably dislodged from the main bed by glacial action.

The Geological Survey Sheet No.69 'Bradford' gives the following succession of rock strata in the area of the proposed development:-

	Rough Rock
Upper Carboniferous (Millstone Grit)	Measures.
	Guiseley Grit.

A number of faults are shown in the area of the site and one is indicated crossing the site in a NNE/SSW direction in the vicinity of the south eastern block, but its line is only an approximation.

A drainage pipe discharges into a rough watercourse in the vicinity of borehole No.3 and runs above ground to a position some 10 yds. SW of the site of borehole No.4 where it appears to go underground to discharge again at a point alongside the canal some 50 yds. SE of the crossing of the Nidd Aqueduct. Excavations at the outlet indicated that this feature may be 'man made' but it was not possible to determine the extent of the construction. The topography of the site suggests that the stone wall to the SE of the northern block is built almost along the line of a fault, along which springs may occur during periods of prolonged rainfall.

The boreholes made encountered 'solid' Carboniferous rocks at depths varying from 6' to 42'6" b.g.l. and in general overlain by clays and sandy clays with sandstone, boulders and cobbles. Boreholes Nos. 2, 6, 6A, 8 and 9 proved gravels and boulders, sometimes in a clay matrix to depths of 42'6", 31', 35', 24' and 17' b.g.l. respectively. These typical glacial moraine deposits occur in the lower parts of the site and tend to 'fall out' to the SE of the site.

Rotary cored drilling proved that the 'solid' rock consisted of brown/yellow competent sandstone in boreholes Nos. 1 and 3, made over the outcrop of the 'Rough Rock'. Other boreholes proved that the rock consisted of relatively thin bedded grey to blue mudstones, sandy mudstones, shales, shaley sandstones and sandstones. No distinctive 'marker horizon' was encountered, but the generally low percentage core recovery and the nature of the cores indicated that weathering was extensive both laterally and in depth. The cores were extensively broken to depth indicating that the rocks possibly lie in a 'fault complex'.

Because of the 'broken' nature of the rock in all boreholes other than Nos. 1 and 3, it was considered that unconfined compression tests on isolated cores would not be indicative of the true strength of the rock mass insitu. Unconfined compression tests on the sandstone from boreholes 1 and 3 gave results of the order of 135/140 tons/sq.ft. Standard penetration tests were made in the clay with boulders and gravels encountered in boreholes Nos. 2, 4, 6 and 9 and gave 'N' values varying from 7 to 36 blows per 12" penetration of the 60° nose probe. The distribution of these results, however, indicated that the density of these deposits varied in depth as well as laterally. Unconfined compression tests and pocket penetrometer tests gave results varying from $\frac{3}{4}$ to $3\frac{1}{2}$ tons/sq.ft.

In boreholes 2, 4, 6 and 9 standing water levels of 12'6", 8'0", 10'0" and 16'0" respectively were observed after the lining tube had been removed from the boreholes. However, in borehole No.11 no water was encountered or observed after removal of the lining tube.

An indication of the generally broken nature of the rock can be obtained by the fact that in boreholes Nos. 3, 5, 6, 6A, 7, 8, 9, 10 and 11 water was either intermittently or completely lost during drilling, and was being discharged along joint planes and fissures in the rock mass.

The National Coal Board stated that no mining of coal has taken place or is envisaged under the site area.

It is understood that this sloping site is to be developed with three multi-storey blocks of flats, and express below opinions on the problems associated with founding such structures at the site. Drawings 26 and 27 indicate that the depth of 'drift' deposits varies over the site area and that local variations may exist between the locations of the boreholes as indicated by rotary borehole No.6A which was made to 35'0" without encountering rock, whilst in borehole No.6, made some 3 ft. away, rock head was encountered at 31'0" b.g.l.

The nature of the drift materials is such that bearing capacities at depths of approximately 6 ft. b.g.l. are of the order of 1 to 2 tons/sq.ft. Isolated base foundations taken to rock would be of the order of from 6 to 43 ft. deep and would encounter water in most excavations. The gravels would probably need to be supported in excavations and when opened out could discharge considerable quantities of water. The broken nature of the rock and its variable distribution both laterally and in depth would indicate that bearing capacities of the order of 3 to 4 tons/sq.ft. should not be exceeded. Mass excavations in rock would probably open up 'springs' along the lines of faults which would flow into the excavations. The evidence of cored boring is such that it is most likely that clay-filled joint planes would be encountered which would require to be raked out and filled with grout.

Mass excavation into the existing slope of the ground for sub-structure construction would be expensive and difficult due to the existence of boulders, rock slabs and water-bearing strata. However, in the vicinity of the northernmost block the drift material is deeper and therefore more easily removable.

The use of piled foundations on this site would also present certain problems notably ensuring that the piles are all founded on the rock-head and not boulders, or slabs of rock. The presence of ground water may also exclude the use of certain types of piles. However, it is considered that piled foundations would ultimately provide the most economical and suitable foundations for this site and the use of large diameter bored piles would avoid closely spaced single pile disturbing boulders whilst adjacent piles are being formed. Large diameter piles would also enable the formation strata to be inspected but the piling contractor's attention should be drawn to the presence of ground water.

Attention is also drawn to the inclination of the 'rock head', which has a maximum fall of 1 in 3. The bearing capacity of piles founded in the rock would be calculated by the formula for deep slender foundations and would probably be in excess of that quoted for isolated bases. The existing drainage should be piped or culverted so that softening of the soils due to increase of moisture content be kept to a minimum.

Chapter 8.

PROBLEMS IN IGNEOUS AREAS.

Igneous rocks pose problems, in that again testing techniques are somewhat unsatisfactory and the geological occurrence of 'pipes' and other volcanic 'irregularities' cannot be determined readily and with any degree of accuracy. Examples of work at Dundee and Greenock are used to illustrate this:

GREENOCK, INVERKIP ROAD. (Drg. No.28)

The site lies on the southern side of a 'glacial valley' and is adjacent to the main Greenock to Largs road. The side of the valley rises quite sharply, as can be seen from the ground contours. The adjacent area to the east and west of the site is developed by 2-storey houses of traditional construction. The cross fall across the site of the block is approximately 10 ft.; below the site the ground falls at a slightly steeper gradient to a burn which runs adjacent to the Largs road.

The proposed development at the site consists of a 15-storey block of flats to house the tenants of the Greenock Corporation.

The Geological Survey Sheet 30 indicates that the site is situated near an outcrop of Macro-porphyritic olivine basalt, formed contemporaneously with deposits of mugearite, which are thought to cover the remainder of the adjacent area. A fault is thought to exist immediately to the east of the site.

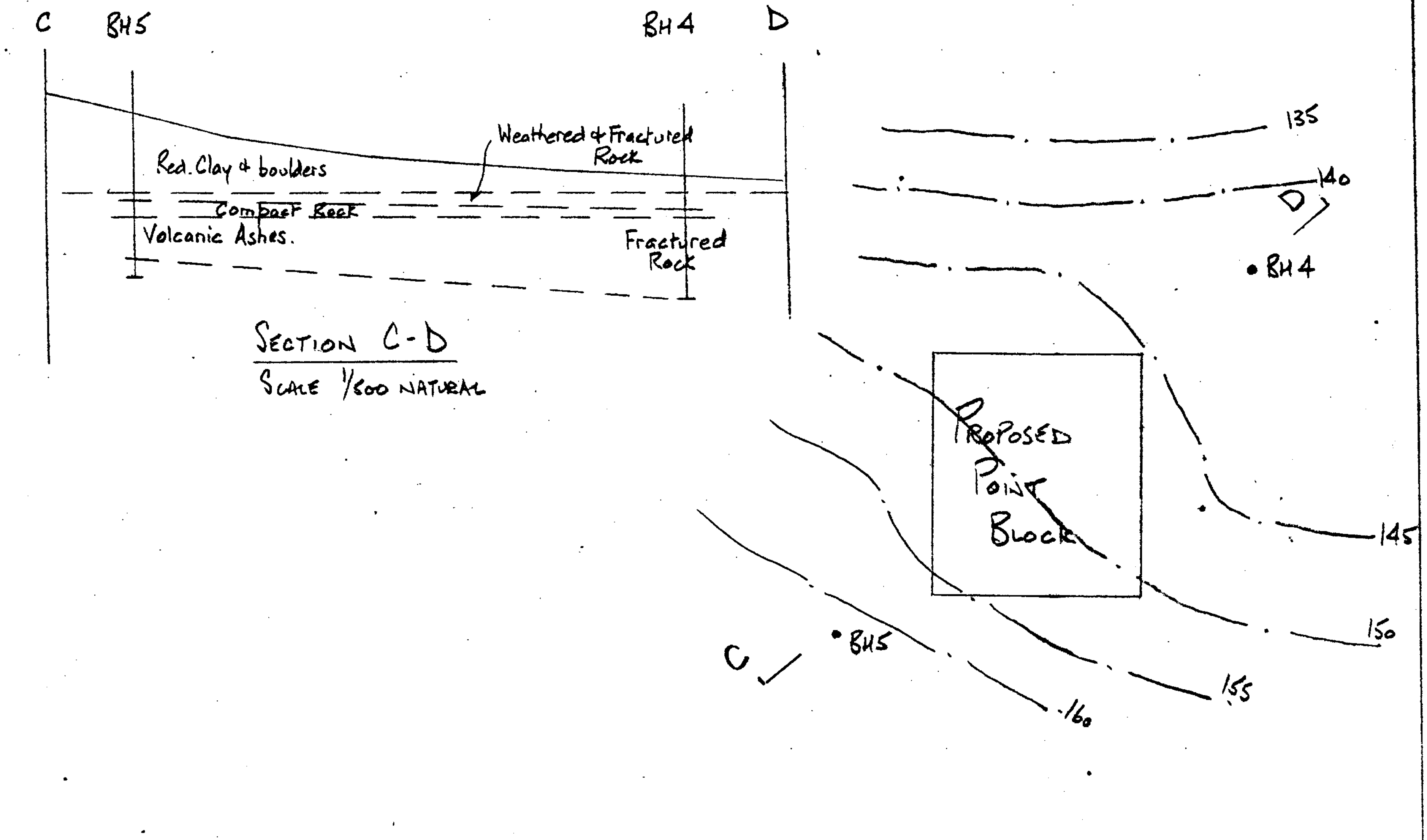
At the instructions of the Consultant, rotary drilling methods were used as it was anticipated that the upper soils were soft and that the loads, due to the proposed structure, would require to be carried on piles taken to rock.

We were instructed to make two boreholes; the first proved red clay with boulders to 5'6" b.g.l. where weathered and fractured red igneous rocks were encountered. Due to the extremely fractured nature of the rock, open hole methods were used until excessive vibration ceased. At 10 ft. a core was taken, but only for a 2 ft. run due to the occurrence of excessive vibration (recovery 56%). The hole continued to 35 ft. but it was not possible to obtain a core. The second hole proved red clay and boulders to 22'6" where igneous rock was struck; from 24 ft. to 29 ft. a 100% core recovery was obtained. From 29 ft. the hole continued in ashes to 41 ft. where rock similar to that at 24 ft. was found; the hole continued to 46 ft.

The section (Drg. 28) indicates the interpretation of the geological structure based on the two holes made. It would appear that the surface of the rock is approximately level and is extensively weathered and fractured. The lateral variations in the lithology are such as could be anticipated in a series of contemporaneous lavas and tuffs. Apart from thin compact rock horizons the strata was found to be intensively fractured (as evidenced by the behaviour of the drilling rig). It is however possible that the site is intersected by a fault.

Samples were crushed by unconfined compression methods; when crushed they disintegrated and produced a residue of ashes. Results varied from 24 to 180 tons/sq.ft. compressive strength.

It would appear that slip due to inclination of the 'rock head' would not be a hazard, and that piles could be founded on the rock as encountered in the boreholes. In assessing the bearing capacity of piles in the upper rock levels the effect of the underlying ashes and fractured rock should be considered. Compressive strengths of 180 tons/sq. ft. were obtained, but the lateral continuation of rock of these strengths cannot be assumed. It



GREENOCK. INVERKIP ROAD.
SITE PLAN & SECTION

DRG. NO. **28**

was recommended that the piles should be keyed into the 'rock', to increase bearing capacity and avoid the hazard of terminating a pile in a boulder.

I understand that work is now proceeding with the block founded on 30" to 36" diameter piles designed to carry loads of from 150 to 200 tons. The piles are to be taken some 15 ft. into the volcanic rock so that the major proportion of the load is taken in skin friction with a nominal end bearing. The making of piles to such depths in rock is of course expensive; generally speaking such conditions of rock and structure as encountered in igneous areas make foundations more expensive than in sedimentary areas. In these areas the problems of differential settlement are greater when loading the foundations to limits associated with multi-storey development. Unfortunately the specialist contractor working to economic limits finds that his degree of confidence in investigations which must be related to his recommendations, is less in the igneous areas than in the sedimentary areas. The economic limits placed on site investigations appear to be fairly standard and do not usually vary on a Regional Geology basis.

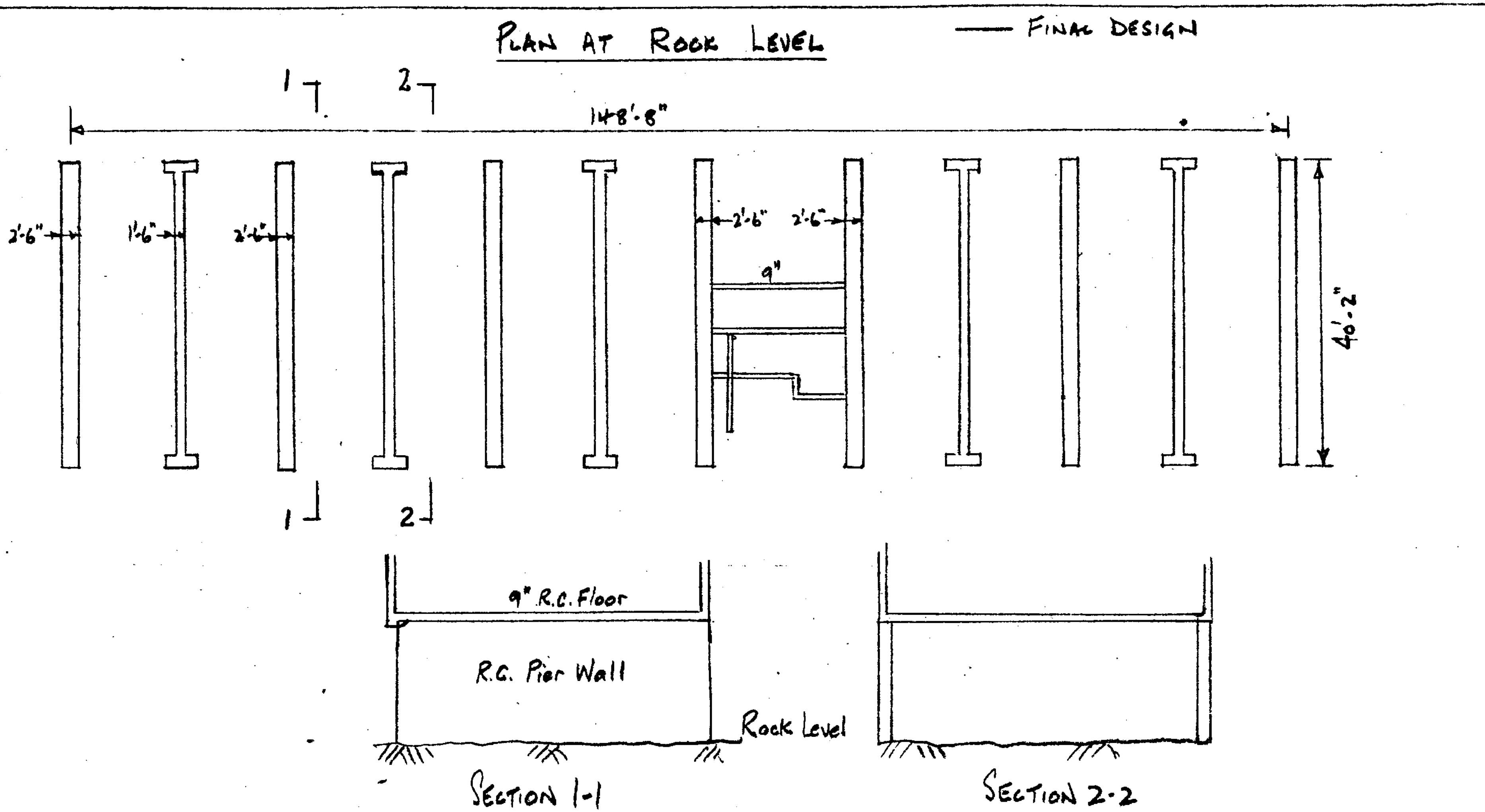
DUNDEE. MAXWELLTOWN C.D.A. (Drg. No.29)

A site investigation consisting of one borehole for an 18-storey block of flats plan area approximately 40 ft. by 150 ft. proved clay and boulders to 12 ft. underlain by some 4 ft. thickness of weathered and open jointed basalt, and then penetrated into sound competent basalt for some 10 ft. The foundations were designed to impose some 10 tons/sq.ft. on strip cross wall construction imposed at the level of the sound basalt. As excavations for the foundations proceeded it became apparent that a volcanic pipe existed in the area of the proposed lift shaft. The pipe was roughly circular and approximately 25 ft. in diameter.

A borehole made from lift shaft formation level proved shattered and weathered basaltic rock to 9 ft. underlain by a 2'6" band of soft red clay; and hence proved amydaloidal basalt in alternating hard and soft layers to a depth of 21 ft. below foundation level. The consulting engineers decided to redesign the foundations to impose approximately 4 tons/sq.ft. on the weathered basalt at formation level.

Whilst only one borehole was made originally and would possibly have been adequate in sedimentary rocks, the consultants were unaware of the possibility of volcanic pipes occurring and had not allowed contingencies against this. Considerable delay and therefore extras to the contract figure were incurred. For future jobs founding in igneous rocks the consultants will allow contingencies for such items but this will not, of course, obviate delays should similar conditions be met. The consultants, on the recommendation of the author, were to approach the Geology Department of Edinburgh University to ascertain whether geophysical work could locate such pipes prior to design.

I understand that to date research being carried out by Manchester University to locate old mine shafts by geophysical methods are far from conclusive, and would suggest that in the case of volcanic pipes, the contrast in physical properties may be no better if not worse than in the case of old mine shafts.



SCALE 20FT TO 1INCH

DUNDEE - MULTI STOREY DEVELOPMENT
PLAN OF FOUNDATIONS

DRG. NO. 29

Chapter 9.

PROBLEMS IN AREAS OF OLD MINE WORKINGS.

Where old mine workings are involved under proposed foundations, the Engineering Geologist closely borders on the field of the Mining Engineer.

Research into subsidence effects have been of great advantage to the Civil Engineering profession. The Mining Engineer has used his practical knowledge allied to theory, to place calculable measurements of settlement, which hitherto had appeared to pose problems which meant that foundation design was virtually beyond solution.

Undoubtedly the Mining Engineer specialising in subsidence has a future in the field of Civil Engineering. Many of Britain's larger cities and towns are underlain by old mine workings (many of which are uncharted), and this, connected to the need for large urban clearance and redevelopment must lead to advances in this field.

The following example from Wigan (Ref. Drgs. 30-37 incl. and Type Records of Boreholes) is typical of this development.

In Coal Measures areas the N.C.B. Deep Mines and the N.C.B. Opencast Executive should be approached to ascertain whether any borings or shaft sections are available. Old shaft sections are often detailed in Memoirs, etc. The interpretation of this information is critical because such borings have usually been made by methods not necessarily akin to those used in Soils Engineering boring. Observations in the boreholes which are of a critical nature to the Geotechnical Engineer have often been neglected as unimportant to the Mining Geologist.

a. WIGAN.

This project was concerned with examining the suitability of building 5 no. 13-storey blocks of flats in an area known to have been extensively worked for coal.

The succession of coal seams in the area consisted of:-

	Approx. interval between seam:
Wigan 5 ft. Seam (outcropping to the east of the site)	
	70′
Wigan 4 ft. Seam	
Trencherbone Sandstone and overlying shales	90′
Wigan 6 ft. Seam	
	300′
Cannel) King)	
	120′
Ravine Mine	
	120′
Yard Mine	

In 1961 a report prepared by Prof. W.G. Fearnsides and the Borough Engineer summarised as follows:-

"The area lying between the River Douglas and School Lane-Warrington Lane can be used for multi-storey development provided the foundations are taken down to beds of sandstone above the Trencherbone Rock Series.

In the case of multi-storey buildings exploratory boring should take place on the individual site prior to preparation of detailed foundation plans."

A number of preliminary borings were made at this stage (called Corporation No. etc.).

Information was also obtained of an investigation made by Geo. Wimpey in the Scholes Park Area which drew the following conclusions:-

1. The Wigan 4 ft. Coal Seam had been extensively worked by pillar and stall methods, and that the workings are presumed to be flooded - (A condition later proved).

2. Although the Wigan 6 ft. Coal Seam was intact in the two boreholes taken to this depth it cannot confidently be regarded as unworked unless its solid condition is also proved in a number of additional boreholes.

My firm were then asked in 1964 to investigate the superficial deposits and rock strata at the site.

First of all, the N.C.B. were contacted and supplied a record plan of workings in the Wigan 6 ft. from nearby Alliance Colliery where the shaft succession was given as:

Ground Level	
	24 ft.
Wigan 4 ft.	
	81 ft.
Wigan 6 ft.	
	285 ft.
Cannel	

A copy of a tracing from this drawing is enclosed (Drg. No.31). There is some suspicion of a minor displacement of the Wigan 6 ft. where the workings are cut off in the area of Block 2. It was also inferred that the Cannel and the Yard Mine had been extensively worked under the site area by pillar and stall methods.

It was now decided to institute the boring programme. Initially the site of Block 1 was investigated because of urgency regarding the commencement of the scheme.

Block 1.

The Corporation investigation indicated that the site of Block 1 lay across the outcrop of the Wigan 4 ft. Coal (which was later proved to have been extensively worked over the whole of the site area). With the exception of borehole 5 near the Old Alliance Pit Shafts, all boreholes which were deep enough proved coal intact at the horizon of the Wigan 6 ft. Coal. It was therefore felt that one of the primary objects of the current investigation was to prove the extent of workings in the Wigan 6 ft. Coal under the site area.

The abandonment plans in the possession of the N.C.B. and the results of the Corporation Site Investigation would indicate that the known workings in the Wigan 6 ft. Coal from Alliance and Birkett Bank Pits are at sufficient distance from Block 1 not to be hazardous.

With reference to workings in seams below the Wigan 6 ft. Prof. Fearnsides commented:-

> "Local surface damage due to workings of these deeper seams is unpredictable and can hardly be insured against."

The Corporation investigation of 1961 proved a tunnel at Scholes Park which conveys pit and possibly other drainage water from an unknown source to presumably outfall in the River Douglas. It was understood that the tunnel runs either across or adjacent to Block 1, but the line is not accurately known. As found at Scholes Park Tunnel it is not lined, and cross sectional dimensions are determined by the amount of overbreak which has occurred along the line of the tunnel where it passes through differing strata. At the site of the block it was anticipated that the depth to invert did not exceed 15 ft. The chemical content of the water being discharged through the tunnel was ascertained so that in the event of a pile intersecting the tunnel the construction is such that the water will not have a deleterious effect on the pile.

Eventually 3 no. piles were made through the tunnel line and were protected by leaving in place the lining tube. Initially three boreholes were made by percussive methods to a depth approximately to the rock head to ascertain the foundation engineering properties of the 'drift' material at the site of Block 1. Boreholes proved soft soils overlying soft coal which is interpreted as the sub-outcrop of the Wigan 4 ft. Coal Seam; proving fill material to a depth of 16 ft. underlain by soft weathered shale; fill and soils of varying density to 12'6" underlain by weathered shale and fireclay presumably just off the sub-outcrop of the Wigan 4 ft. Coal.

Standing water levels of 1'10", 9'2" and 2'6" were proved in the boreholes and any relationship to the level of water in the River Douglas was not proved.

In view of the variable density and lateral distribution of the drift and fill material, and the high water table, it was considered that this material was not suitable to carry foundations to multi-storey blocks irrespective of the condition of solid formations below and the extent of coal workings.

Three wash boreholes were made to prove the existence of cavities in the rocks at the site. None of these boreholes proved cavities either above or at the horizon of the Wigan 6 ft. Coal. One borehole gave no return of water from 85'0" to 100'0" but no cavities were found.

Two cored boreholes were made to the underside of the Wigan 6 ft. Coal which was proved to be intact; cavities above this horizon were not proved.

The cores from both boreholes were not extensively broken, but did break off along certain bedding planes and exhibit some degree of almost vertical jointing as one would anticipate in competent Coal Measures strata of this nature. One hole proved the sub-outcrop of the Wigan 4 ft. Coal at shallow depth but this was not encountered in the other borehole. Both boreholes proved mudstone and sandy mudstones overlying the Trencherbone Rock. The comparison of these two cored boreholes together with an interpretation of dip of the strata over the adjacent area, indicates that either some rapid lateral facies variation or faulting has taken place at the site of Block 1. Both boreholes lost water at 63'6" and 64'5", possibly on the suspected fault plane.

Strike lines for the Wigan 6 ft. were drawn for the area and it was strongly suspected that a fault running roughly north/south lies at the eastern side of Block 1 at the horizon of the Wigan 6 ft., and has a minor throw to the east. The significance of the fault is not thought to be critical from the point of subsidence but may have some local effect on the length of piles. (The geological structure is discussed later at greater length.)

Assuming that the tunnel invert was approximately 15 ft. below ground level (as discussed previously) the effect of a piled foundation is that should the tunnel be intersected by a pile the construction should be such that the water does not have a deleterious effect on the pile.

It was not anticipated that any old shafts exist under the site but the possibility cannot be precluded. In view of the aforementioned, it was recommended that the structure be founded on a piled foundation bearing on competent Sandy Mudstone strata below the soft fireclay underlay to the Wigan 4 ft. seam. This fireclay is associated with a thin coal seam below the Wigan 4 ft.

It may be considered advantageous to move the site of Block 1 off the outcrop of the Wigan 4 ft. Coal so that some savings due to lesser difficulties of driving, and in pile length, may be made. Because of the effect of dip and strike it was considered that a bored pile would be advantageous so that the lithology of the rock can be checked in the individual pile. The ground floor slab should be designed as fully suspended.

Blocks 2 to 5 incl.

Originally 14 no. boreholes were made by percussive techniques and insitu tests were made by the standard penetration method.

In soils such as those encountered (i.e. clays and sands with stones and boulders, etc.), the standard penetration test was considered to be the most practical to obtain readings which are indicative of bearing values.

Two types of rotary drillings were also used to investigate the rock strata to depth, these being to obtain knowledge of the strata sequence and to probe for cavitation.

Cored holes were made, and as some reductions in the coring tools became necessary the core diameters obtained were $2\frac{5}{8}$" - $2\frac{3}{8}$" and $2\frac{1}{8}$". Representative core samples at the critical horizons were prepared and tested unconfined to compression failure in a testing machine of 200 tons total capacity.

Wash-boring techniques were the second type of drilling employed, these being used as an economical and relatively quick comparator to the cored holes. Observations were also kept to record loss of drilling water as a possible guide for any consideration that may be given to methods of stowing cavities.

The investigation of the remaining 4 no. blocks then proceeded. In the first instance a further 7 no. cored holes were made at the sites of the various Blocks. After an analysis of the information obtained at this stage, it became evident that extensive working of the Wigan 4 ft. seam had taken place and therefore that in view of the relatively shallow depth of the seam sand stowing of cavities appeared to have economic and engineering advantages.

The services of a drilling contractor with experience in sand stowing techniques were engaged, and 28 no. wash bores were then made over the complete site, including the British Railways property opposite the site of Block 2. Certain of these bores could not penetrate to the level of the Wigan 6 ft. seam due to considerable broken rock strata, but those holes made in areas which were previously thought to be relatively undisturbed penetrated to this horizon; and it was felt that the ease of penetration was indicative of the degree of disturbance first encountered.

Interpretation of the bore logs progressed with the work on site and it became apparent that one or two gaps in the overall picture would arise, and to avoid this it was then considered necessary to make a further 4 no. cored boreholes. It should be mentioned in this instance that it was not always possible to drill in the ideal position due to the existence of properties.

To summarise the drift and fill material: at the sites of Blocks 1, 2 and 4, nearest to the River Douglas, the soils are soft and water bearing, whilst at Blocks 3 and 5 the soils are relatively compact and do not contain water in observable quantities.

Drawing No.34 illustrates the thickness of the drift and fill material overlying Coal Measures strata.

GEOLOGICAL SUCCESSION.

The succession varies in detail across the site, and the general succession at each block is as follows:-

Block Two	Drift and fill	13' to 22' thick
	Broken shale	0' to 4' thick
	Wigan 4 ft. Coal Seam in 2 leaves -	

Block Two (Cont.)	Upper Leaf av.	3'9" Thick
	Parting	2'9" Thick
	Lower Leaf	1'11" Thick
	Soft Fireclay	12' to 16' Thick
	Mudstone	14' to 22' Thick
	Sandstone	44' to 50' Thick
	Mudstone and Sandy Mudstone	13' to 15' Thick
	Wigan 6 ft. Coal Seam	8'2" to 8'8" Thick
<u>Block Three</u>	Drift and Fill	10' Thick
	Sandstone	0' to 10' Thick
	Sandy Mudstone	0' to 11' Thick
	Broken Shale	12' to 15' Thick
	Wigan 4 ft. Coal Seam in 2 leaves:	
	Upper Leaf	2'10" Thick
	Parting	2'5" Thick
	Lower Leaf	1'6" Thick
	Soft Fireclay	12' Thick
	Mudstone	6' to 15' Thick
	Sandy Mudstone	15' Thick
	Sandstone	37' to 48' Thick
	Mudstone and Sandy Mudstone	22' Thick
	Wigan 6 ft. Coal Seam	8'6" to 9'2" with thin partings.
<u>Block Four</u>	Drift and Fill	7' to 16' Thick
	Sandstone	10' Thick
	Soft broken Shale & Fireclay (possible collapse into workings in Wigan 4 ft. Coal Seam)	7' Thick

Block Four (Cont.)	Mudstone	11' to 12' Thick
	Sandstone with thin Mudstones & Sandy Mudstones at Upper Levels	38' to 45' Thick
	Mudstone	14' to 15' Thick
	Black Shale	1' Thick
	Wigan 6 ft. Coal Seam	8'6" to 8'9" Thick
<u>Block One</u>	Drift & Fill	11' to 16' Thick
	Wigan 4 ft. Coal Seam	Outcropping
	Fireclay & Shale	9' to 10' Thick
	Mudstone and Sandy Mudstone	29' to 44' Thick
	Sandstone	0' to 10' Thick
	Sandy Mudstone	0' to 6' Thick
	Sandstones	6' to 15' Thick
	Sandy Mudstone	5' to 8' Thick
	Mudstone	6' to 11' Thick
	Black Shale	1' to 2' Thick
	Wigan 6 ft. Coal Seam	7'11" to 9'4" Thick
<u>Block Five</u>	Drift & Fill	11' to 12' Thick
	Shales	0' to 10' Thick
	Sandstone	2' to 5' Thick
	Broken Shales & Mudstones	0' to 20' Thick
	Broken Mudstones & Sandy Mudstones	0' to 12' Thick
	Broken Black Shale	7' to 12' Thick

Block Five (Cont.)	Wigan 4 ft. Coal Seam - Upper Leaf	1'3" to 3'8" Thick
	Parting	1'0" to 2'11" Thick
	Lower Leaf	1'9" to 2'1" Thick
	Shale & Fireclay	10' to 13' Thick
	Mudstone	0' to 10' Thick
	Sandy Mudstone	5' to 36' Thick
	Mudstone	0' to 6' Thick
	Sandstone	6' to 21' Thick
	Mudstone and Sandy Mudstone	6' to 29' Thick
	Sandstone	0' to 7' Thick
	Mudstone and Sandy Mudstone	16' to 20' Thick
	Black Shale	0' to 10' Thick
	Wigan 6 ft. Coal Seam	7'11" Thick where not worked.

The principal feature of a comparison of variation in the succession is the facies variation in the Trencherbone Sandstone from thick sandstone under Blocks 2, 3 and 4, feathering out with inclusions of sandy mudstone towards Blocks 1 and 5.

The logs of the Corporation boreholes and Messrs. Wimpey boreholes were incorporated in the interpretation of the geological structure.

The method of interpretation was to plot strike or 'contour' lines on the base of the Coal Seams. In Coal Measures strata it is considered that the only suitably reliable horizons for such interpretation are the coal seams, and the base was selected as being the most positive horizon of the coal seam to identify.

The interpretation is shown on Drawings Nos. 33, 35 and 36. The general dip is comparable for both seams and varies from 1 in 5.5 to 1 in 9 in the Wigan 4 ft. Coal Seam; and from 1 in 5.5 to 1 in 7.5 in the Wigan 6 ft. Coal Seam. The direction of dip is fairly constant at about 15 to 20^{o} N of E.

Faults have been indicated on the drawings in positions where the variations in the strike lines indicated some displacement. At the lower horizon, i.e. the Wigan 6 ft. Coal Seam, the interpretation of the boreholes at the site of Block 2 is only thought to be satisfactorily resolved by the presence of a reverse fault. It is interpreted from the details of workings in the Wigan 6 ft. seam from Alliance Colliery, that some major disturbance has taken place at this location. Several long probing galleries are shown on Drawing No.31 which possibly were driven to investigate the strata in advance of working. It is possible, bearing in mind the difficulties encountered in drilling certain holes, that the 'reverse fault' continues in that direction, being approximately parallel to the present course of the River Douglas.

The Wigan 4 ft. Seam is thought to outcrop as indicated on Drawing No.35 and therefore intersects the sites of Blocks 1 and 4.

To summarise, the site is intersected by a number of normal faults of varying downthrow, and bounded to the west of Block 2 by a major disturbance thought to be a reverse fault. The coals dip generally east, but this is in the same direction as the rise of the ground surface, therefore they become deeper at the sites of Blocks 3 and 5 than at Blocks 1, 2 and 4. The Wigan 4 ft. Coal Seam outcrops across the sites of Blocks 1 and 4, and is at shallow depth below the drift at the site of Block 2.

EVIDENCE OF MINE WORKINGS & 'BROKEN GROUND' (Related to solid drilling)

Drg. No.37 illustrates the occurrence of voids and 'broken ground'. The table has been prepared so that all boreholes are indicated using a datum of the base of the Wigan 6 ft. Coal Seam or where the horizon could be anticipated with reference to Drawing No.36.

The boreholes made at Block 1 did not give any evidence of major disturbance and one small area in Wash 2 may be associated with a fault zone.

At Block 2 the broken ground at depth in borehole Core 5 may be associated with subsidence from the adjacent Alliance Colliery workings in the Wigan 6 ft. as also is probably the case in boreholes Wash 5, 10, 12 and 15 and Core 13. The void encountered in Wash 14 is associated with the workings in the Wigan 4 ft. whilst those in Wash 6 and 10 are associated with workings in the Wigan 6 ft. seam. It would appear, therefore, that a certain amount of subsidence due to working in the Wigan 6 ft. seam has already taken place but has been restricted by the presence of the Trencherbone Sandstone; this however also could be due to the existence of a reverse fault.

At Block 3 the broken ground at high level is probably associated with workings in the Wigan 4 ft. Coal Seam whilst those at depth in Core 4 (and in the Trencherbone Sandstone) are probably associated with subsidence from lower seams; the extent in depth of the fractured zone is greater than would be anticipated due to a fault zone.

At Block 4 the presence of broken ground at high levels can only be attributed to subsidence from lower depths than the Wigan 4 ft. Seam. This may be due to a continuation of Alliance Colliery workings in the Wigan 6 ft. Seam from the area adjacent to Block 2.

At Block 5 workings in the Wigan 6 ft. Coal Seam were proved in boreholes

Core 9, 10 and 11, and the occurrence of broken ground is as would be anticipated. It appears however at this site that the sandstones and sandy mudstones, which are the equivalent of the Trencherbone Sandstone in other areas, has effectively to date acted as a beam over the workings. Broken ground at higher levels is undoubtedly associated with workings in the Wigan 4 ft. Coal Seam; whilst those areas below the level of the Wigan 4 ft. Coal may be associated with the trough faults thought to exist under the site of the Block.

All cored boreholes to adequate depth, made by this investigation, with the exception of Nos. Core 9, 10 and 11 and Wash 6, proved the full thickness of Wigan 6 ft. Coal to be intact, as did all Corporation boreholes and Wimpey boreholes of adequate depth. The Wigan 4 ft. Coal is thought to have been worked under the whole area probably by pillar and stall methods. This may have stopped at the fault running N-S shown on Drawing No.35, but shallow 'pirate' workings to the west of the fault may have taken place during the Depression and General Strike (1925-28). The Wigan 6 ft. Coal is known and proved to have been worked from Alliance Colliery adjacent to Block No.2 and also up to the major fault under Block No.5 shown on Drawing No.36.

<u>EVIDENCE OF MINE WORKINGS & BROKEN GROUND</u> (Related to loss of drilling water)

Observations were made during drilling of return of the water used as a lubricant. General description of water losses at the site of each Block is given below:-

<u>Block One</u>

Both core boreholes lost water immediately above the Trencherbone Sandstone; this did not return but lining tube was not taken to this depth. Wash bores 1 and 2 suffered no loss of water for the complete length of hole; the other lost water at 85 ft.

Block Two

Core boreholes Nos. 5, 7 and 13 did not lose circulation water until the Trencherbone Sandstone was penetrated; this loss was presumably along joint planes in the rock. Wash boreholes Nos. 4, 5, 6 and 7 lost water at depths below 48 ft., 37 ft., 38 ft. and 37 ft. respectively, but this returned as boring proceeded, and in No. 6 even when the void was penetrated, return of water continued. Wash borehole No. 9 did not lose water until 105' b.g.l. and this returned at 115' b.g.l. Wash borehole No.10 lost water in a void at 50' b.g.l. but this returned when drilling proceeded to 56' b.g.l.; however water was lost at 90' b.g.l. which did not return for the full length of the hole (118'). Wash boreholes Nos. 11 and 12 suffered temporary losses of water as drilling proceeded. Wash borehole 15 lost water intermittently at higher levels and 10 g.p.m. for $\frac{1}{2}$ hour at 70' b.g.l. It should be noted in connection with the wash boreholes that the casing in most holes, unless otherwise noted, was only through the drift material.

Block Three

Core borehole No.4 lost water in broken ground above the Wigan 4 ft. Coal Seam but this was returned, to be lost again in the coal seam. Core borehole No.15 did not lose water in broken ground associated with the Wigan 4 ft. Coal Seam but water was lost in the Trencherbone Sandstone. Wash borehole No.23 suffered partial loss of water at 17 ft. and at 93 ft. to lose water completely at 109 ft.

Block Four

Both core boreholes lost water in the Trencherbone Sandstone. Wash borehole No.20 did not lose water until the Trencherbone Sandstone. Wash Borehole No.21 did not lose water to 50 ft. b.g.l. Wash borehole No.22 suffered temporary losses of water, and No.24 lost water at 68 ft. b.g.l.

Block Five

Cored boreholes lost water at and above the Wigan 4 ft. seam and in the equivalent of the Trencherbone Sandstone. No return was observed when drilling below the Trencherbone Sandstone, but of course casing did not penetrate to this depth.

Wash borehole No.28 lost appreciable quantities of water immediately below the Wigan 4 ft. Seam. Wash borehole No.29 lost water completely in the void of old Wigan 4 ft. Seam workings, as did No.30 .

Wash borehole No.31 lost water both at and below the level of the Wigan 4 ft. Seam. No loss of water was observed in wash borehole No.27.

Boreholes in Low Street between Blocks 2, 3 and 4.

Wash boreholes Nos. 16 and 17 proved voids at and above the Wigan 4 ft. Seam horizon coupled with loss of water. Holes Nos. 18 and 19 lost water intermittently in the lengths of the holes.

Boreholes in Garden Street adjacent to Block 5.

Wash borehole No.23 suffered no loss of water whilst No.26 suffered temporary losses.

EVIDENCE OF SURFACE SUBSIDENCE.

A 'line of break' exists across the site of Blocks 2 and 4, in a line drawn from Withnall Street to Scholes indicated by a recent subsidence during the time of this investigation; strutting between houses; strapping of houses; cracking in houses; tilted lintel, etc. At the recent subsidence (below the gable end foundations of No.1 Withnall Street) a probe was made to 18'8" b.g.l. before refusal was met. This subsidence appeared overnight and the surface effect was of the order 14 ft. x 5'6" x 12 ft. deep.

The trend of this 'line of break' is parallel to the fault in the Wigan 4 ft. shown on Drawing No.35, and also approximately parallel to the line of disturbance projected in a southerly direction. As the lintel above the door to No.8 Withnall Street is tilted to the east, it is thought probable that the subsidence is due to workings in the Wigan 4 ft. Seam, but it is possible, however, that this effect is cumulative, due also to workings in the Wigan 6 ft. Seam.

SULPHATE CONTENT.

Samples taken from percussive boreholes were despatched to an Analytical Chemist who performed tests to ascertain the sulphate content of soil and water. It will be noted that the soil samples contain sulphates in negligible proportions, but that all the water samples tested contain sulphates in proportions lying in Class 2 of the 'Classification of Sulphate Soil Conditions affecting concrete', published by the Building Research Station.

GENERAL OBSERVATIONS ON CAVITY FILLING TECHNIQUES.

The following techniques of cavity filling were then considered:

(a) Sand Stowing

Boreholes are made on a grid pattern (say 15 ft.) over the site to be stowed. These holes are drilled by open hole methods to the depths of the voids. A mixture of sand containing a low percentage of clay in suspension in water is placed down the borehole commencing at the dip side of the area to be stowed, thus forming an underground dam. As the mixture enters the void the particles in suspension precipitate out, and are deposited in the void. Sand stowing on each individual hole continues until no further material can be placed. A number of relief and check holes are also made.

No specification of the particle size distribution of the material to be used is thought to exist; the material being selected principally on a basis of experience.

Any pricing of sand stowing requires an estimate of the size of the grid of boreholes, the depth of the void remaining and the percentage extraction in the coal workings.

Doubts regarding the adequacy of this technique are related to:-

(i) the lateral extent of the voids

(ii) the extent of effective stowing in height of the voids and density of the fill

(iii) the effect of underground water flowing in the workings and subsequently eroding the stowed material. The velocity of water flowing would be difficult to estimate due to the roughness of the debris on the floor of the workings and the almost incalculable hydrostatic head and therefore the hydraulic gradient. It may be however that the water pressures operating on the sand stowed material could have the effect of 'piping'.

(iv) The presence of silty debris at the seam pavement.

(b) Infilling of cavities by 'grouting' techniques.

In this method the area to be treated is determined by a $1:\frac{1}{3}$ ratio (depth from foundation to pavement, to lateral extent) outside the area of the foundation.

Initially 4" diameter holes are made at 5' centres on the dip side and two adjacent sides of the area to be treated. A three-sided dam is created by a mixture of sand, fly ash and a little cement which is placed down the hole to form a 'truncated cone' in the open working. The floor debris is first grouted to ensure that the floor material is made competent. As the 'cone' reaches the top of the working it is rammed and further grout placed in a repetitive sequence. Practical tests made indicate that the cone assumes an angle of 40° to the horizontal. Adjacent cones are made at 5' centres and then the remaining void between cones is spray grouted to complete the dam.

The infilling inside the dam is made through 2" diameter holes at 10' to 15' centres, dependent on the collapse condition, and height of the working. The fill consists of a sand, fly ash, and mixture of cement (approx. 120 lbs. per cu. yd. of infill). Further injection is then carried out if necessary to tighten up the fill material. It is considered that the strength of the infill is from 120 to 150 p.s.i.

If grouting techniques are used in water filled voids then a dry concrete is used to allow for increase in moisture content of the concrete. Strengths of concrete can be varied by mix but it is doubted whether, under the water conditions, 750 p.s.i. can be obtained. Filling is effected by special grout tubes designed to discharge so that water has a nominal effect. If the seam has collapsed it is not possible to form a barrier wall dam as described above. In this event a 'mushroom technique' is used where the borehole is made to seam pavement and grout is injected to form 'inverted saucer' grouted areas. This is allowed to set and further successive 'saucers' are built up.

Doubts regarding the adequacy of these techniques are related to:

1. Extent of effective stowing and the precise strength of the grouted area.
2. Filling under water - difficulties would appear to be compatible with sand stowing.
3. The presence of silty debris at pavement level.

(c) Grouting of rock strata.

Another technique used to stabilize ground and fill cavities is that known as grouting. In this process injection boreholes are made, again on a grid pattern, and then cement or chemical grout is injected into the strata to be stabilized. It is my opinion that none of the rocks existing at the site are of adequate porosity to allow grout to penetrate. In the case of the Trencherbone Sandstone, however, it is felt that grout would penetrate along the joint planes where water was lost during drilling of the holes made in this investigation.

FOUNDATIONS

The following recommendations were based on the borehole data on examination of samples, and the results of site and laboratory tests.

Whilst no old mine shafts were encountered during the investigations the possibility of these being encountered during excavations and in piling cannot be precluded. The following covering clause was included in the report: "The interpretation of the geological structure has been made in good faith but naturally the accuracy is proportional to the number of boreholes and the availability of sites for drilling."

A close drilling pattern based on sizes of pillar and stall workings to prove the Wigan 6 ft. Coal was envisaged at one time, but in view of the number of holes required over the site, and the limited access, was not thought to be practical.

DISCUSSION OF FOUNDATIONS

(a) Block One

The structure be founded on piles taken to the mudstones and sandy mudstones which lie below the Wigan 4 ft. Coal Seam and Fireclay. In view of the high water table and the sulphate content, it was also recommended that the piles and the suspended ground floor be constructed with concrete made with sulphate resisting cement.

(b) Block Two

The preliminary boreholes gave conflicting results and therefore more than the average number of boreholes were made.

(a) Drift - loose made-up ground from 10 to 16 ft. b.g.l. with soft soils below, underlain by Coal Measures Strata.

High Water Table in the order of 6 ft. b.g.l.

(b) Soft broken shale in areas isolated by faults - approximately 4 ft. thick.

(c) Wigan 4 ft. - at shallow depth below the base of the drift material. Depth of seam 23 to 30 ft. Core hole No.5 and Wash bore No.15 showed broken ground above the horizon of the Wigan 4 ft. Coal.

(d) Fireclay - consistency not proved but possibly soft.

(e) Mudstones and Sandy Mudstones - broken ground was proved in boreholes Wash 10, 12, and Core 13, above the Trencherbone Sandstone.

(f) Trencherbone Sandstone.

(g) Wigan 6 ft. Coal - Wash bore No.6 proved a 48" high void at this horizon but Wash bore No.4 and Core holes Nos. 5, 7 and 13 proved the full thickness of the coal seam.

Wash bores Nos. 8 and 11 encountered coal at depths not compatible with either the Wigan 4 ft. or Wigan 6 ft. Coal Seams. Sections were set up and the only apparent solution to satisfy the conditions of geological structure was to assume that a reverse or thrust fault occurred under the site. This explanation would substantiate to a certain degree the evidence that workings in the Wigan 6 ft. (an apparently profitable seam) were terminated at such a short distance from the shafts. It is possible also that water is entering the workings along the plane of the fault.

<u>Possible Solution of Foundation Problem</u>.

Due to the complex geological structure, the probability that the Wigan 6 ft. workings only partially extend under and adjacent to the site; and because the draw effect may be aggravated by the reverse fault, two possible solutions were offered:

(a) Bulk excavation to remove the deep fill, soft drift material, and the then exposed remains of the Wigan 4 ft. Coal Seam. As these workings would be adjacent to the River Douglas, the Crompton Street bridge abutments, and the main thoroughfare; and because of the soft soils and high water table, the excavation would require to be enclosed by sheet piling. In order to obtain a cut-off, sheet piles would need to be driven some distance into the fireclay. The top of the sheet piling can only be supported by cross strutting.

Due to any possible inconsistencies in the fireclay and future settlement due to movement of the broken ground, a stiff raft designed for mining subsidence should then be provided. The moisture content and therefore shear strength of the fireclay would vary quickly when exposed to sub-aerial weathering agencies, and therefore provision should be made to blind the exposed surface immediately this horizon has been reached. The joints in the Trencherbone Sandstone should be filled by grouting techniques, and any cavities in the broken ground immediately above this horizon and below raft foundation level, filled by sand stowing techniques. This solution would of necessity be expensive.

(b) The alternative is to grout the Trencherbone Sandstone and fill any cavities in the broken ground and Wigan 4 ft. Seam by sand stowing techniques.

The grout injection and sand stowing holes should be made on such a pattern that they lie directly on the same grid points as the pile positions, which should be taken through the fireclays to the mudstones and sandy mudstones below. At this horizon the piles could be designed to impose a safe bearing of 10 tons/sq.ft. (which for end bearing piles was suggested can be multiplied by a coefficient of 5 to allow for lateral confinement due to the overburden pressure).

It was not possible to make tests on samples of the fireclay but the opinion was expressed that a shear value of 3 tons/sq.ft. could be assumed in calculating skin friction.

The ground floor should be designed as fully suspended.

In view of the high water table and sulphate content, the piles and suspended ground floor would require to be constructed with concrete made with sulphate resisting cement concrete.

(c) Block Three.

(a) Drift - areas of filled ground to approximately 2 ft. to 3 ft. 6 ins. deep, soils capable of bearing capacity of $1\frac{1}{2}$ tons per sq. ft. at levels varying from 110.62. The number of percussive boreholes and the siting was restricted by the existing property. However, the penetrometer tests made at the two boreholes showed that the upper soils are loose and it is reasonable to assume that existing foundations and service runs have caused disturbance. The sandstone appears to be at approximately 10 ft. deep but possibly outcrops across the site of the Block.

(b) Broken from underside of drift to Wigan 4 ft. Coal Seam.

(c) Wigan 4 ft. partially or completely worked, depth of seam 33 to 47 ft.

(d) Fireclays, mudstones and sandy mudstones.

(e) Trencherbone Sandstone - broken in Core borehole No.4.

(f) Wigan 6 ft. not proved to be worked, depth of seam 133 to 150 ft.

Possible solution to Foundation Problem.

A stiff raft could be provided at a depth of approximately 6 ft. at the lowest part of the site (subject to further short percussive boreholes being made after demolition of the existing property), provided that sand stowing techniques or similar are used to fill the cavities and broken ground down to the Wigan 4 ft. This broken ground will vary in depth below the raft due to the dip of the Wigan 4 ft., and any minor faults not disclosed during this investigation. The Chemist did not report any excessive sulphate content for the soils below this Block, and as a water table was not encountered, it was considered that normal portland cement could be used in the raft.

(d) Block Four.

(a) Drift - Borehole Perc. 9 showed fill down to 9'0" b.g.l. with a further 3 ft. of soft soils below. Boreholes Perc. 7 and 12 showed very loose and disturbed soils to 10 ft. Borehole 8 showed fill to 3 ft. with firm soils below. High Water Table approximately 4 ft. below the lower ground level.

(b) Wigan 4 ft. - on the evidence of the limited drilling possible, due to existing property, it would appear that the Wigan 4 ft. Seam outcrops below the drift within the N.E. corner of the site of the Block. Core 3 encountered soft shales and stones which may be old workings in the Wigan 4 ft. Coal Seam.

(c) Boreholes Wash 20 and Wash 24 are similar in that they did not prove broken ground whilst Wash 20 found solid coal at the level of the Wigan 6 ft. However, Core boreholes Nos. 3 and 8 proved broken ground to depths above the principal leaf of the Trencherbone Sandstone (44'6" and 34'4" respectively). It is possible that this broken ground may either be associated with the faulting shown on Drawing No.36, or by the existence of cross faults located on either side of the block (but not shown on the drawing because they could not be substantiated); or due to the extension of the 'reverse fault' thought to be adjacent to Block 2 but not proved by the limited drilling pattern. As a result of the latter alternatives the broken ground immediately under the block could be the result of extensive draw effect (from the workings in the Wigan 6 ft. from Alliance Colliery) which has been limited between minor faults.

(d) Trencherbone Sandstone - Lower Leaf.

(e) Wigan 6 ft. not proved to be worked - depth of seam 95 to 115 ft.

Possible Solution of Foundation Problem.

Bulk excavation to remove the deep fill and soft soils. The site is adjacent to the River Douglas and the use of sheet piling may be necessary due to the existence of soft soils and the high water table. Difficulty would be involved, however, in obtaining a cut-off for sheet piles due to the occurrence of sandstones over the east part of the site, immediately below the depths at which water was encountered in the percussive boreholes.

Any cavities in the broken ground and ? Wigan 4 ft. workings above the principal leaf of the Trencherbone Sandstone could be filled by sand stowing techniques, and a very stiff raft designed for mining subsidence principles could then be provided.

In view of the high water table, and sulphate content, it was recommended that the piles and suspended ground floor be constructed with concrete made with sulphate resisting cement concrete.

(e) Block Five.

(a) Drift - areas of filled ground to approximately 8 ft. deep; soils capable of $1\frac{1}{2}$ tons/sq.ft. bearing capacity at levels varying from 116 to 121' O.D. No ground water encountered.

(b) Broken from underside of drift to Wigan 4 ft.

(c) Wigan 4 ft. worked and shattered. Depth of seam 40 to 55 ft.

(d) Sandstones, sandy mudstones and mudstones which have suffered irregular deposition, probably associated with the penecontemporaneous movement of the trough fault indicated in the Wigan 6 ft. Coal Seam.

(e) Broken ground below base of (d) due to workings in the Wigan 6 ft. Coal Seam.

(f) Wigan 6 ft. A trough fault is thought to exist under the Block. Workings have taken place from the south, probably terminating at the fault.

Possible Solution of Foundation Problem.

A stiff raft foundation could be provided at a depth of approximately 8 ft. below existing ground provided sand stowing techniques or similar used to fill the cavities and broken ground down to the Wigan 4 ft.

The Chemist did not report any excessive sulphate content for the soils below this Block, and as a water table was not encountered, it was considered that normal portland cement could be used in the raft.

The following conclusions were drawn:

(a) The main problems in designing adequate foundations at this site are subsidence due to the collapse of underground coal workings and the fact that in the area of Blocks 1, 2 and 4 the Coal Measures strata is overlain by appreciable depths of soft water-logged fill and soils.

(b) Attention was drawn to the fact that this investigation and the resulting foundation recommendations were based on the assumptions and conclusions given in the report prepared by Professor Fearnsides and Mr. Keighley, the Borough Engineer and Surveyor. For this reason, deep drillings were not made to prove the condition of the ground below the Wigan 6 ft. Coal Seam.

(c) Under most conditions of surface subsidence, stiff rafts are considered to offer the most suitable foundation.

At Blocks 2 and 4 the possibility of further subsidence from collapse of old workings in the Wigan 6 ft. cannot be ignored; unfortunately, however, at these sites the depth and nature of the drift is such that piling provides the most apparently economical solution to convey bearing loads to horizons capable of supporting those loads. This foundation is, of course, more susceptible to movement than the raft, and the use of grouting and sand stowing techniques was suggested at levels not directly in and above known workings, in order to stabilize the upper ground to provide a 'sill' of uncavitated areas of sufficient depth to absorb laterally any collapse of the lower workings.

(d) It was suggested that in the case of Blocks 2 and 4 the final analysis should be made after a study of the relative costs, and that the services of a Consulting Mining Engineer be engaged to comment on the possibility of further subsidence occuring.

After discussions with the Mining Consultant and the Structural Design Engineer the Consultant reported that as the minimum depth of the Wigan 6 ft. seam is 70 ft. and because of the presence of the massive sandstone (Trencherbone Rock) he was satisfied that there was no risk of pillar failure in the seam nor of collapses to the surface into any voids which may exist at or immediately above the seam level. Even if piled foundations were taken to below the 4 ft. seam, providing they did not penetrate more than a nominal distance into the Trencherbone Rock, no treatment of ancient workings was necessary. The presence of the Trencherbone Rock above the 6 ft. seam and below the possible foundation level for the blocks would distribute the loads. Any question of 'draw' at Blocks 2 and 4 does not therefore arise, and the proposal to grout the Trencherbone rock is therefore not applicable. The Wigan 4 ft. workings, however, constitute a serious instability problem, and it was therefore recommended that a shaft be made to enable visual inspection of the working. Unfortunately the shaft did not reach the 4 ft. level due to water under pressure being encountered (this eventually reached a rest level of 102.5). This meant that the Consultant was forced to assume extreme conditions in assessing differential settlement and the dimensions of crown holes, both factors which influence the raft foundation design proposed for Blocks 3 and 5. It was accepted in the cases of Blocks 1, 2 and 4 that piled foundations would be most suitable, as there did not seem to be any instability problem due to subsidence at these sites.

However, in the case of Blocks 3 and 5, he recommended that there appear to be sufficient measures above the 4 ft. seam to allow sand stowing and that uniform load distribution through a raft foundation is best in conjunction with a stabilisation programme.

One problem associated with piling in such ground arises primarily because blocks of rock may be disturbed in the process of driving, which could result in lateral thrusts being exerted on the piles with the possibility of shear. It would appear that large diameter piles should be seriously considered.

It had been decided to pile Block 1, and work was proceeding on site before approaching the Consultant. His comments generally confirmed the original recommendations. The pile groups consisted of 3 to 4 no. 21" dia. piles each designed to carry a load of 60 tons, which is equivalent to 25 tons/sq.ft. end bearing. Piles were driven to depths of from 25 to 29 ft. below existing ground level so that by reference to the borehole logs for the area they penetrated approximately 7 to 16 ft. into the Coal Measures strata. Adequate site supervision was provided to assure that piles did not terminate on soft rock. Piles were made by driven insitu techniques with a bulb driven at the base of the pile. It is not thought, however, that any bulb was created at the site (as determined by the volume of concrete used).

The Consultant's recommendations were accepted for Blocks 2 and 4, but for Blocks 3 and 5, where rafts were proposed, he was asked if he could determine the size and dimensions and centres of 'crown holes' which would enable a raft to be designed to cope for all conditions of span and cantilevering effects. In connection with Block 5 he suggested that as the thickness of Coal Measures strata lying between the Wigan 4 ft. seam and the base of the superficial deposits varies from 9 to 35 ft. across the proposed site, the collapse of strata spanning remnant pillars was of importance. Obviously this condition is of more consequence where the cover is least, because possible resultant differential settlement would be undesirable. The

Consultant also reported that the incidence of such areas of settlement was completely unpredictable, but that as the cover increased the risk of settlement of the surface decreased.

It was recommended that the Block be sited some 50 ft. to the east where surface effects of old mine workings would be less critical and where no remedial measures (such as sand stowing) would be required. It was suggested that if the Block could not be moved, then the old workings be filled with sand plus some setting material. However, due to the variable thickness of competent measures, some compaction of the fill material could occur, with resultant differential settlement. Sand fill holes were proposed on a square grid pattern at 20 ft. centres with test holes at 10 ft. centres diagonally between. The cover of proposed holes extended some 40 ft. outside the outline of the block on the 'rise side' and 60 ft. on the 'dip side'.

It was also suggested that the cost of this work plus the cost of raft would make the movement of the Block an economic proposition. It was agreed that the block be moved and shell and auger holes were made to prove that a raft imposing $1\frac{1}{2}$ tons/sq.ft. on the superficial soils was feasible.

b. OLD MINE SHAFTS - example DEWSBURY.

The location of disused mine shafts on development sites has been a problem and one which is still largely unsolved. The necessity for proving the locations of such mine shafts is principally concerned with total collapse of backfill material either due to the increase in imposed loading or to unpredictable settlement of the fill. Shafts vary in size and may be backfilled with spoil or capped with timber, old rails, or reinforced concrete slabs, etc. What are the practical applications of such a problem? The following example illustrates some of the problems posed by old mine shafts.

The inquiry for site investigation at Dewsbury indicated the locations of old shafts as obtained from the N.C.B. Drillings carried out at the locations of the shafts did not prove their existence. It was appreciated that the shaft locations were probably slightly in error, and a 40 ft. square grid of holes @ 5 ft. centres was drilled from the presumed shaft locations without success.

A meeting was then held with the Client to discuss the necessity of proving the shafts. It was considered that there are two ways of dealing with the problem. Firstly, to prove shaft locations and hence design the individual foundations to allow for the presence of shafts, and secondly, to build every foundation so that the existence of a shaft would not be a hazard wherever they may exist. The above proposals involve economic comparisons and in the case in question this was analysed as follows:-

Site area approximately 25 acres.

(a) A programme of boring at 5 ft. intervals would be exorbitant and was priced at approximately £165,000. Such a programme would give complete coverage.

(b) Electrical resistivity survey using Wenner configuration with two probe readings at probe spacings of 20 to 40 ft., to be taken at 20 ft. intervals over the entire area. Approximately 2700 test points would be required for full coverage. Anomalies would need to be checked by drilling.

Resistivity survey	£1500
Boreholes	700
	£2200

Success could not be guaranteed, and it is understood that comparable work carried out by Manchester University at Skelmersdale has met with only limited success.

(c) Comprehensive trench excavation over the site was considered, but was felt to be impractical as foundation bearing could be ruined and at depths of say 4 ft. may not disclose shafts capped at lower levels.

(d) Design all foundations as R.C. strip footings without location of shafts. Strips provided to span an 8 ft. void and also to cantilever 8 ft. at the ends of the houses would require reinforcement of the order of $\frac{1}{2}$ ton per house.

Design, supply and fix reinforcement
per block of 2 houses (semis) .... say £70

If the area is 25 acres and density
is 14 houses per acre this would cost £12,250

In this appraisal it is assumed that settlement of an isolated floor slab of maximum area 14' x 12' x 5 inches thick could be tolerated. However, if the floor slab was tied to the walls, a greater margin of safety could be achieved and this method would assure complete success assuming that any area of collapse was not greater than 8 ft. dia.

A comparison of the costs indicates that efforts to prove shafts are economic if kept within reason.

Air photographs of the area obtained by the client indicated approximately 15 "shadow anomalies" at the site. Assuming that each of these was investigated using a 40 ft. grid at 5 ft. centres, this would cost approximately £9,000.

Where one draws the limit is a matter for the Client to decide, having been provided with an assessment of the overall problem.

Chapter 10.

CONCLUSIONS.

1. The Geologist and the Engineer both have roles to play in the investigation of sites and the preparation of geotechnical reports, as no well-marked demarkations exist. Whilst the Civil Engineer has taken the major role in the development of geotechniques, methods and interpretations used have led of necessity to the creation of the 'Geological Engineer' and the 'Engineering Geologist'. Good geologist engineer relations exist in organisations where foundation design forms an appreciable part of the work. The geologist, however, should realise fully that foundation design depends on a good knowledge of the design of the structure to be built upon those foundations. It is the attitude of the Civils that an engineer versed in soils is not fully equipped unless he can design the foundation. Whilst appreciating this very valid point, it is the author's experience that the engineer's appreciation of Geology is not always so wide. The significance of this appreciation is apparent when one considers that the number of tests made per unit area is relatively small and that geological correlation is an essential part of investigation.
2. The responsibility for site investigation is in the hands of the Employer (i.e. Engineer, Architect, Local Authority, etc.) who usually exercises little or no control over techniques used on site. The future must lie with drilling and laboratory contractors under the control of geotechnical engineers who are employed either by the contractor or the client. Some consultants have intimated to me that only professional conduct and

presumably liability dissuades them from running their own drilling rigs and testing laboratories in order to facilitate better investigation. Most papers on this topic refer to quality, which unfortunately in most instances is only as good as the economic situation allows. In this condition the specialist contractor must comment if he feels that the volume of work is inadequate; this of course implies that he is thoroughly conversant with the client's requirements and that the site investigation is closely related to design procedure.

Duncan, in his series of papers entitled "Geology and Soil Engineering" published in the "Muck Shifter" 1965, states:-

"A site exploration programme done without meticulous attention to detail is better left undone. It can give the Engineer a false sense of security, or alternatively lead him into designing against dangers which are non-existent.

..... The imperfections in the system of competitive tendering may be reflected in the finer points of geological significance. It is such small geological details which in many cases govern the nature and extent of the difficulties and problems of sub surface engineering. That confidence can be built up between a client, be he consulting engineer or main contractor, and one or more of the specialist contractors of his own choosing has been proved beyond doubt."

Whilst agreeing almost entirely with Duncan, I am a little dubious that any specialist contractor is ever beyond doubt, as indeed would be anyone in the sphere of geotechnical engineering. Too many features such as technique, supervision, and beyond all interpretation, make it virtually impossible that any sub surface specialist be infallible.

Duncan goes on to say that:-

"Engineering Geology and soil mechanics are indispensable geotechnical partners in programmes of soil exploration and testing. Considered together they ensure that the overall picture of sub surface conditions as well as detailed quantitative data, is appreciated and available to aid in the design, planning and construction of works."

It is interesting to note that Hammurabi (2000 B.C.) King of Babylonia must have had an appreciation of the effects of these problems when he had the following Code of Laws prepared: (translated by R.F. Harper)

"a. If a builder build a house for a man and do not make its construction firm, and the house which he has built collapse and cause the death of the owner of the house that builder shall be put to death.

b. If it cause the death of the son of the owner of the house, they shall put to death a son of that builder.

c. If it cause the death of the slave of the owner of the house, he shall give to the owner of the house a slave equal value.

d. If it destroy property, he shall restore whatsoever it destroyed and because he did not make the house he built firm and it collapsed he shall rebuild the house which collapsed at his own expense.

e. If a builder build a house for a man and not make its construction meet the requirements, and a wall fall in, that builder shall strengthen the wall at his own expense."

Whilst our Codes are somewhat more tolerant, liabilities are covered to a great degree, especially in site investigation where no liability for opinions (given in good faith) required to provide a comprehensive report, is given.

3. Research work in the field of Rock Mechanics is primarily in its infancy

but would appear to have a greater application in large civil engineering projects than to building structures. My discussions with engineers have led to the belief that in general works increase of bearing capacities in excess of 10 tons/sq.ft. for isolated base foundations, and say 40 tons/sq.ft. for deep cylindrical foundations, would be of little direct economic advantage. In shallow foundations there is a limiting size in which excavation can proceed and hence tends to determine the width of footings and hence imposed loading on the soil or rock.

4. The principal guides to Engineering Geology are the Civil Engineering Codes of Practice. When dealing with rocks, the Codes are principally descriptive and it is the author's opinion that where softer rocks are involved, much is left to be desired. It is not always considered economical to take cores on every site, so one is left much to the driller's description of the rock as taken from the return drilling fluid or the chippings. The extent of jointing, soft partings, dip, etc., cannot therefore be adequately allowed for and so conservative appreciation is necessary in view of one's responsibility.

5. In soft rocks it is felt that piling calculations compared to driving resistances can be informative. Until recently the piling contractor with his fund of experience has largely been vague in his discussions on calculation of bearing capacity. This was no doubt due partly to his fear that the engineer would too literally translate theory into practice. However, from the opposite view it would appear that the piling contractor has not in the past taken full advantage of correlating his experience with theory. With the advent of the augered and bored pile (where no driving resistance is calculable) he has had to lean more on quantitative studies. It is interesting to note in this context that I have recently

heard that one major piling contractor has made available to his soils engineer his complete records for analysis. No doubt this is a mammoth task, but one which will benefit all, if the results are published.

6. It is my opinion that research should be made into loading tests to failure on piles and shallow bases in rocks of all types. Test results could then be correlated with soil and rock studies. I know of one consulting engineer who invariably tests a pile to failure on each of his sites, and then quantitatively analyses the results against known formulae. This is considered to be of great value but is largely due to the good offices of his Client.

7. I have attempted to indicate that test procedures in soft rocks are suspect and suggest that research be made into the suitability of the insitu pressuremeter as a testing tool. Gibson and Anderson in their paper "Insitu Measurement of Soil Properties with the Pressuremeter" (Civil Engineering, May 1961) produced results which would encourage belief in the suitability of this apparatus to fill a much-needed gap.

   The Schmidt Rebound Hammer has been used to discriminate between hardnesses in rocks, and it is thought possible that the use of the tool could be of advantage in classifying and testing soft rocks.

8. I am firmly of the opinion that in view of the number of site investigations made in most city areas, and the necessity to satisfy the Building Surveyors, that all available information should be correlated for each specific area on a rational basis. This would be principally a task for the engineering geologist, as distribution of soils and rocks is in his direct field of application. The situation arises where investigations are made by different contractors on behalf of different clients on

adjacent sites. I am sure that large authorities (who hold copies of all investigations in their area) would find it of advantage to have a 'master plan' of their area. Planning Engineers could then be made generally familiar with anticipated soil and rock conditions, and overall planning be made on a more rational basis. On certain sites the need for site investigation in detail is questionable if no claims are to be entertained for delays or for redesign of foundations. From the 'master plan' (exercised with judgment and caution by a geotechnical engineer) one could obtain a reasonably accurate idea of conditions which could be anticipated, and the designer could proceed with reasonable confidence.

Where 'package deals' are concerned, these are usually on a fixed price basis and foundations either priced on assumed ground pressures or on the results of a site investigation. The geotechnical engineer should be used to full advantage before making a site investigation, i.e. by making visits to research libraries, N.C.B. records, etc. Advance information can lead to more efficient and in many cases more economical investigations. This technique is operated by a few authorities and contractors, but in the case of the latter this is not always compatible with economic running.

9. Researches into the effect of subsidence due to old mine workings is considered essential. It is the author's experience that conflict of opinion exists between mining engineers as to how subsidence occurs and what form it takes. One can sympathise with the mining engineer because he is as appreciative of the vagaries of nature as is the geologist.

The civil and structural engineer, however, requires quantitative information to enable him to design foundations.

I would suggest that future lines of research be made to study:

a. The efficiency of cavity filling techniques.

b. Conditions under which failure occurs in pillar and stall areas. The designer when considering a raft foundation needs an indication of the diameter and centres of 'crown holes' which could occur beneath the raft.

10. The broad descriptive terms of 'Engineering Geology' require some quantitative measure attached to them. Whilst being appreciative of the difficulties involved in relating the 'science to the art', it must be borne in mind seriously that liberal thought in interpretation based on experience should not be stimulated by the conservatism and reticence of others.

REFERENCES

1. Civil Engineering Code of Practice No.1. 'Site Investigations' London. Inst. Civ.Eng. 1950.

2. Civil Engineering Code of Practice No.4. 'Foundations' London. Inst. Civ. Eng. 1954.

3. Nelson A. (1965) "Dictionary of Mining" London Newnes.

4. Legget R.F. (1939) "Geology and Engineering" McGraw-Hill Book Company Inc.

5. Seelye E.E. (1956) "Foundations, Design and Practice. New York Wiley 1956.

6. Feld J. (1965) "Failures in Foundations. Wisconsin . Soiltest Inc. (1965)

7. Tomlinson M.J. (Sept.1961) Symposium on Large Diameter Bored Piles. The Reinforced Concrete Review Vol.V.No.11.

8. Birch N . (1965)"Engineering Properties of Keuper Marl. Proc. Geol. Soc. No.1621 Pub. 11th June, 1965.

9. Jessop R. H. B. and Eastwood W. (1964) 'Effect of shaft friction on the carrying capacity of the base of a pile or foundation. Civil Engineering and Public Works Review Nov. 1964.

10 Woodward H.B. (1881) Memoirs of the Geological Survey of the County of Norwich. London Geol. Survey (1881)

11 Golder, H.Q. and M. W. Leonard 1954 "Some tests on bored piles in London clay" Geotechnique, 4: 32-41.

12. Meyerhof G.G. and L. J. Murdock. 1953. "And investigation of the bearing capacity of some bored and driven piles in London clay." Geotechnique. 3: 267-282.

13. Moran D. & G. S. Jain (19631) "Bearing capacity of bored piles in expansive clays." Proc. 5th Int. Conf. Soil Mech., 2: 117-121.

14. Skempton A.W. (1959) "Cast in-situ bored piles in London clay."Geotechnique. 9: 153-173.

15. Tomlinson M.J. (1957) "The Adhesion of piles driven in clay soils" Proc. 4th Int. Conf. Soil Mech., 2:66-71. Tomlinson M.J. (1958) ("A note on piles in sensitive clays" Geotechnique. 7:4:192. Dr. Golder,) Correspondence on Paper, Geotechnique, 8:187.

16. Whitaker T. and R. W. Cooke (1961) "A new approach to pile testing." Proc. 5th Int. Conf. Soil Mech., 1961 2:171-176.

17. Whitaker T. and R. W. Cooke (1961) "Experiments on model piles with enlarged bases." Geotechnique 11:1-13.

18. Williams G.M. J. (1957) "Design of the foundations of the Shell Building, London." Proc. 4th Int. Conf. Soil Mech. 1: 457-461.

19. Fleming W.G.K. and W. W. Frischmann (1960) "Loading tests on an under-reamed bored pile." Civ. Eng. and Publ. wks. Review. 55:650:1151-1152.

20. Duncan N. (1965) "Geology and Engineering" London. 'Muck Shifter.' Grampian Press.

21. Gibson R.E. and Anderson W.F. (1961) "Insitu Measurement of Soil Properties with the Pressuremeter." London. Civil Engineering Public Works Review. May 1961.

22. Kolek J. (1958) "An appreciation of the Schmidt rebound hammer" Mag. Conc. Research Vol.10. No.28 March 1958.

23. Wardell K. and Wood J. C. (1966) "Ground Instability Problems arising from the presence of old, shallowmine workings." Proc. Midland Soil Mech. and Found. Eng.Soc. Vol. 7. 1966. Paper No.36.

24. Meyerhof G.G. (1956) "Penetration Tests and Bearing capacity of Cohesionless Soils." Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers. No.SM1.

25. Green H. (1961) "Long term loading of short bored piles." Geotechnique Vol.11. 1960-61.

Acknowledgements.

Acknowledgements are gratefully extended to the Liverpool Corporation (Structural Engineers Dept.) Truscon Ltd., Rowlinson Constructions Ltd., the City of Norwich (City Engineer H. C. Rowley M.I.C.E.,) David Balfour & Sons M.M.I.C.E., Consulting Engineers, F. W. Woolworth & Co. Ltd., (Chief Architect D. W. Hardy Dip. Arch., A.R.I.B.A.) The Corporation of Greenock (Burgh Architect & Town Planning Officer, J. Watson D.A. A.R.I.B.A. A.M.T.P.I.,) The Corporation of Wigan (Engineer & Surveyor G. Keighley, B.Sc., A.M.I.C.E., M.I.Mun.E.) and the Bingley UDC (Chief ArchitectS.M. Arnfield, L.R.I.B.A., A.M.T.P.I.,) for permission to illustrate this thesis by case examples of site investigation.

I would also thank all former colleagues for the experience I have gained by discussion of Engineering geological problems, and in particular Mr. T. C. L. Trafford A.M.I.Struct.E., F.G.S., of Truscon Ltd.,

ERRATA

Page 1. Line 13 Should read ..... the engineer did not have supreme Faith in the Art;...

14. Line 1. for Tomlinson read Whitaker.

Page 19. Line 19 Should read .... Standard penetration tests.....

Page 25. Line 8 Should read ....A theretical cohesian value of 0.20 tons/sw.ft. was adopted....

Page 30 lines 9, and 21 ... instead of unit of soil .... read unit wt. of soil.

Page 31 Line 14 ... formula should read

$$qf = K \gamma_D NqA.$$

Page 39. Line 10 Should read ..... It is the considered opinion of many Engineers.......

REFERENCES.

1. Civil Engineering Code of Practice No.1 (1950) 'Site Investigation'.
2. Civil Engineering Code of Practice No.4 (1954) 'Foundations'.
3. Dictionary of Mining, A. Nelson.
4. Geology and Engineering. Leggat.
5. Foundations, Design and Practice. E. E. Seelye.
6. Failures in Foundations. J. Feld, Soiltest Inc.
7. Symposium on Large Diameter Bored Piles, reported in The Reinforced Concrete Review, Vol. V, No.11, September 1961.
8. 'Engineering Properties of Keuper Marl', Birch; Proc. Geol. Soc. No.1621, 11th June 1965.
9. 'Effect of shaft friction on the carrying capacity of the base of a pile or foundation'; Jessop and Eastwood, Civil Engineering, November 1964.
10. Memoirs of the Geological Survey of the County of Norwich; H.B. Woodward, 1881.
11. 'Some tests on bored piles in London Clay'; Golder and Leonard. Geotechnique, 4, 1954, pp.34-41.
12. 'An investigation of the bearing capacity of some bored and driven piles in London Clay'. Meyerhof and Murdock; Geotechnique 3, 1953, pp.267-282.
13. 'Bearing Capacity of bored piles in expansive clays'. Moran and Jain. Proc. 5th Int. Conf. Soil Mech. 1961.
14. 'Cast insitu bored piles in London Clay'. Skempton, 1959. Geotechnique 9.
15. 'The adhesion of piles driven in clay soils'. Tomlinson. Proc. 4th Int. Conf. Soil Mech. 1957.

16. 'A new approach to pile testing'. Whitaker and Cooke, 1961. Proc. 5th Int. Conf. Soil Mech.

17. 'Experiments on model piles with enlarged bases'. Whitaker and Cooke 1961 Geotechnique 11.

18. 'Design of the foundations of the new Shell Building London'. Williams, 1957 Proc. 4th Int. Conf. Soil Mech.

19. 'Loading tests on an under-reamed bored piles'. Fleming and Frischmann 1960 'Civil Engineering'.

20. 'Geology and Soil Engineering'. Neil Duncan, 'Muck Shifter', 1965.

21. 'Insitu Measurement of soil properties with the Pressuremeter' by Gibson and Anderson. May 1961 'Civil Engineering'.

22. 'An appreciation of the Schmidt rebound hammer' by Kolek, Mag. of Concrete Research, Vol.10, No.28, March 1958, pp.27-36.

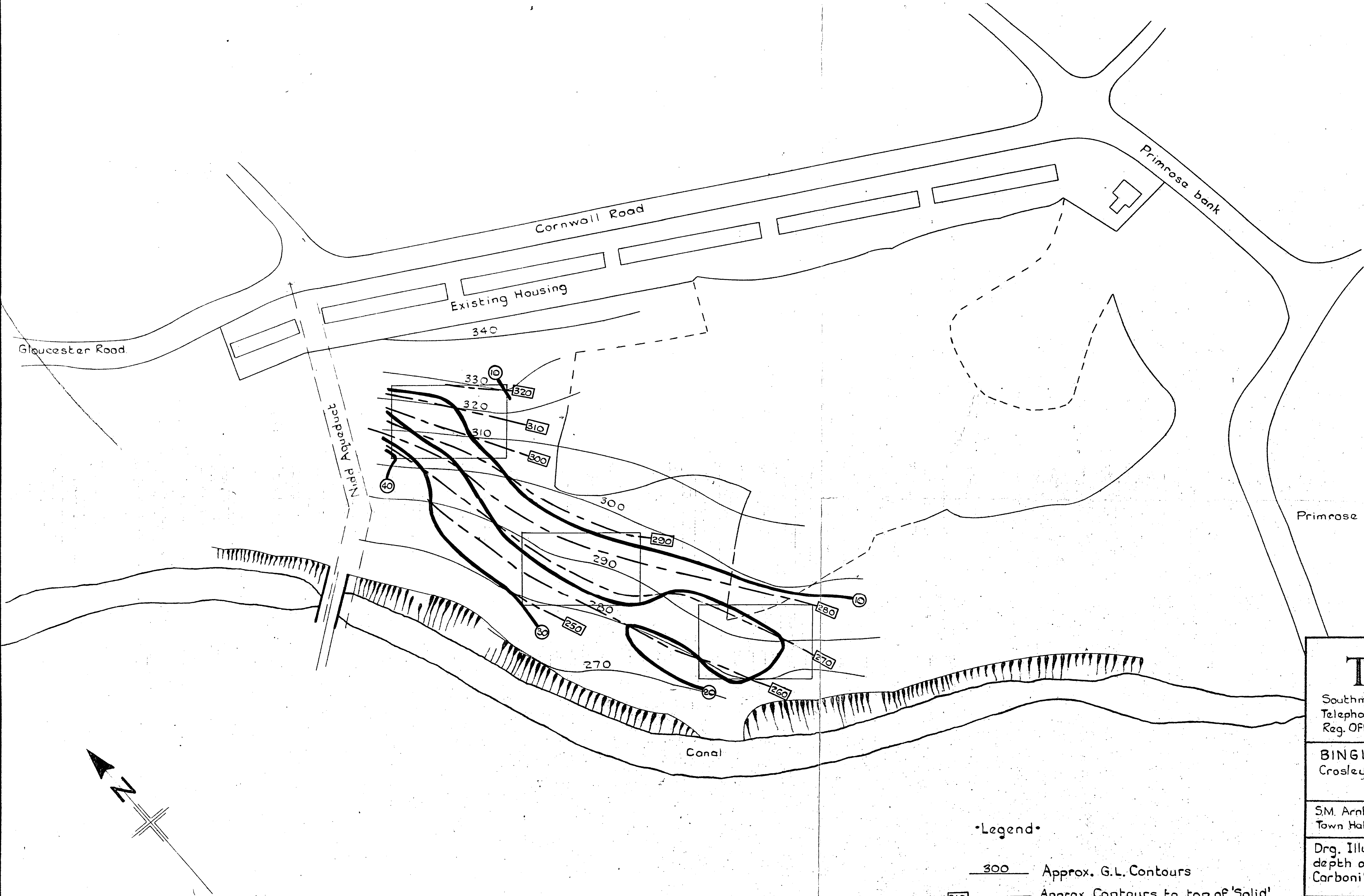
23. 'Ground instability problems arising from the presence of old, shallow mine workings' by Wardell and Wood; Proc. of the Midlands Soil Mech. and Foundation Engineering Society Paper No.36, Vol.7, 1965.

24. 'The recording and interpretation of geological conditions in the foundations of the Roseires, Kariba, and Latiyan Dams'. Knill and Jones, Geotechnique Vol.XV, No.1, 1965.

25. Meyerhof. Journal of Soil Mechanics and Foundations Division. Proc. A.S.C.E., Vol.82, January 1956.

Acknowledgments.

Acknowledgments are gratefully extended to both the City of Liverpool Structural Engineers' Dept. and to Truscon Ltd. for their collaboration and permission to review work carried out by the author on their behalf.



·Legend·

300 Approx. G.L. Contours

310 — — Approx. Contours to top of 'Solid' Carboniferous Rocks (e.g. shales, mudstones, sandy mudstones and sandstones etc.)

10 ——— Isopachyte of thickness of 'Drift' deposits overlying carboniferous rocks.

Truscon LIMITED

Southmoor Road, Wythenshawe, Manchester 23.
Telephone Didsbury 7771/6
Reg. Office - 35-41 Lower Marsh, London S.E.1.

BINGLEY
Crosley Wood Development Phase 2.

S.M. Arnfield Esq. Dip.T.P., A.M.T.P.I. Architect,
Town Hall, Bingley, Yorks.

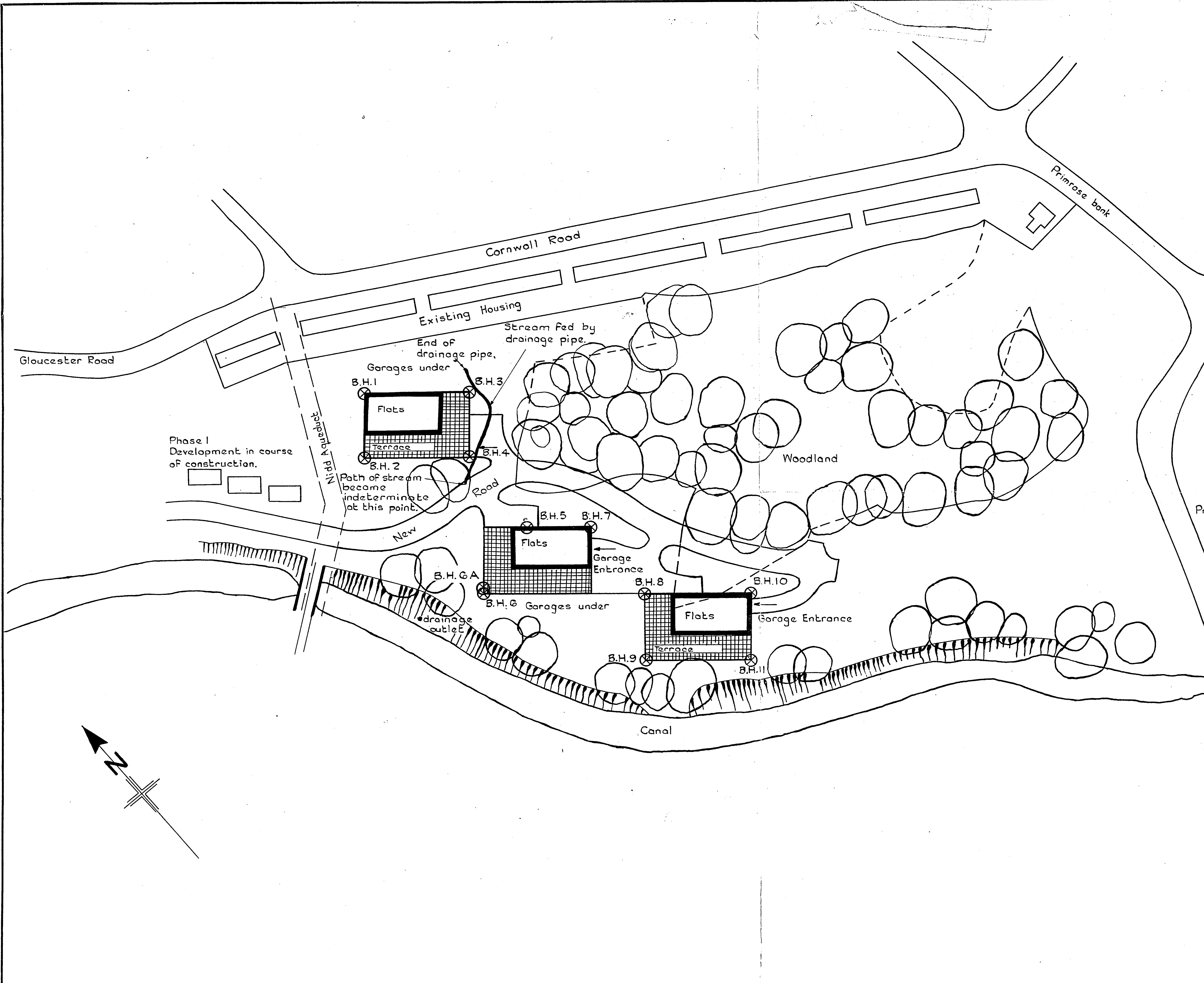
Drg. Illustrating Relationship of G.L. to depth of 'Drift' deposits overlying 'Solid' Carboniferous Rocks.

SCALE: 1/1250

JOB No. SN 964		DRAWING No. B	
REVISION			
DATE			
SIGNATURE	D.Bly.		

25

LR 6319



Truscon LIMITED
Southmoor Road, Wythenshawe, Manchester 23.
Telephone Didsbury 7771/6
Reg. Office - 35-41 Lower Marsh, London S.E.1.

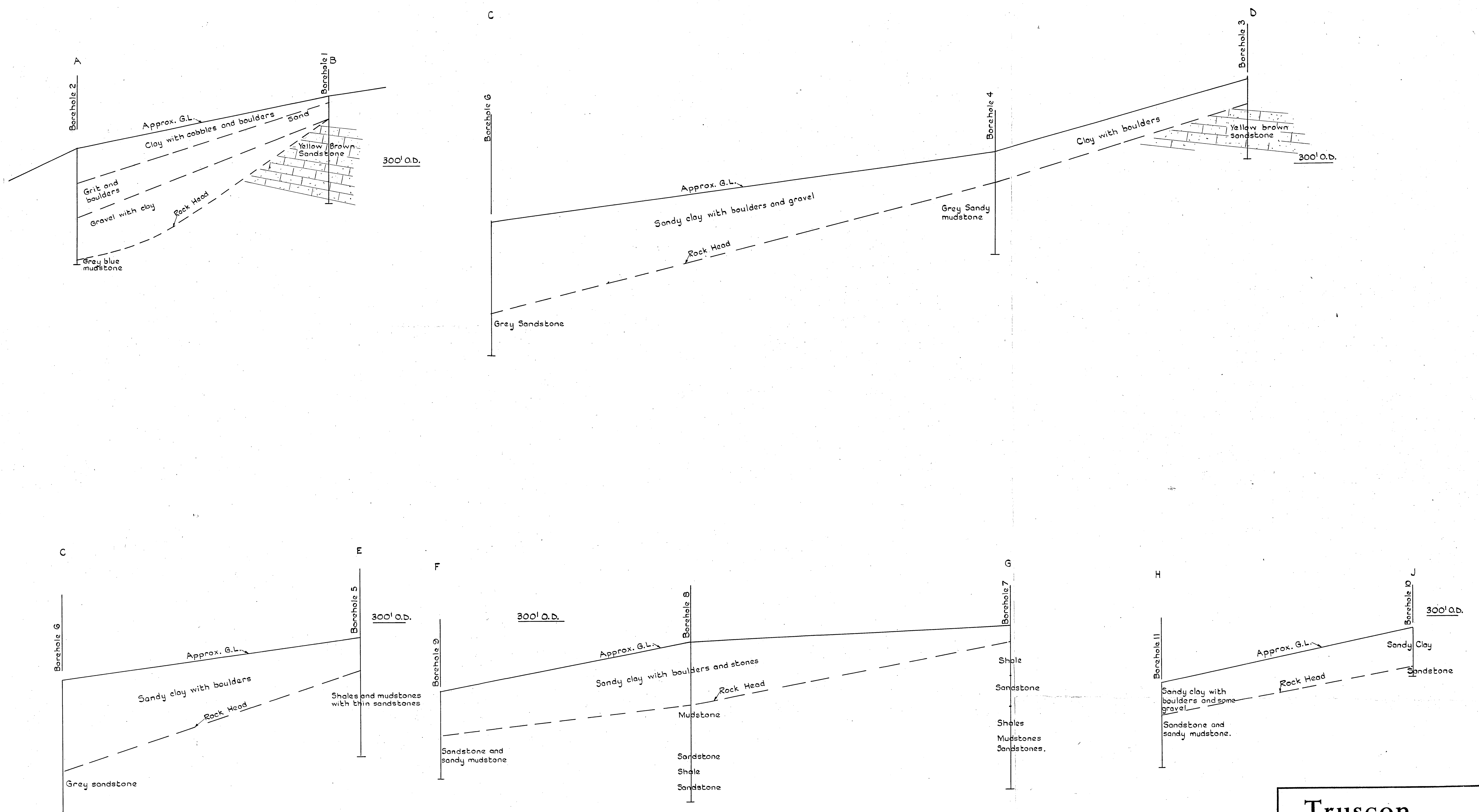
BINGLEY
Crosley Wood Development. Phase 2.

S.M. Arnfield Esq. Dip.T.P., A.M.T.P.I. Architect,
Town Hall, Bingley, Yorks.

Site Investigation
Site Plan

SCALE: 1/1250		
JOB No. SN 964		DRAWING No. A
REVISION		26
DATE		
SIGNATURE	A.B.Ley.	

LR 6819



Truscon LIMITED

Southmoor Road, Wythenshawe, Manchester 23
Telephone Didsbury 7771/6
Reg. Office: 35-41 Lower Marsh, London S.E.1.

BINGLEY
Crosley Wood Development. Phase 2

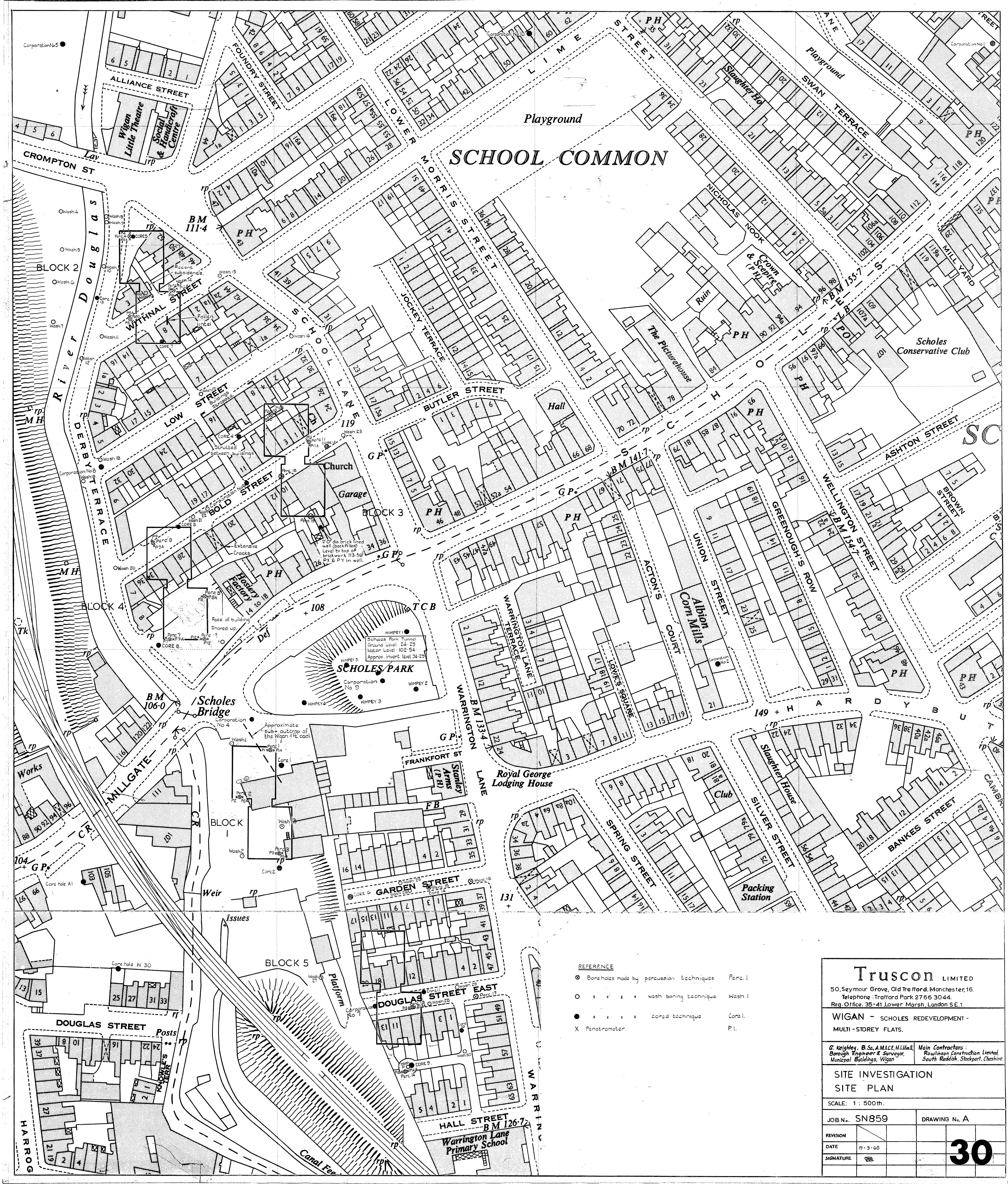
S.M. Arnfield Esq. Dip.T.P. A.M.T.P.I. Architect, Town Hall, Bingley, Yorks.

Sections Illustrating Distribution Of 'Drift' Deposits.

SCALE: Natural Scale 1" = 20Ft.

JOB No. SN 964		**DRAWING No.** D	
REVISION		**27**	
DATE			
SIGNATURE	D.E.Ley		

LR 6818



REFERENCE

⊗ Boreholes made by percussion techniques		Perc. 1
O " " " "	wash boring technique	Wash 1
● " " " "	cored technique	Core 1.
X Penetrometer.		P. 1.

Truscon LIMITED
50, Seymour Grove, Old Trafford, Manchester, 16.
Telephone : Trafford Park 2766 3044.
Reg. Office. 35-41, Lower Marsh, London S.E.1.

WIGAN - SCHOLES REDEVELOPMENT - MULTI-STOREY FLATS.

G. Keighley, B.Sc., A.M.I.C.E., M.I.Mun.E., Borough Engineer & Surveyor, Municipal Buildings, Wigan	Main Contractors : Rowlinson Construction Limited, South Reddish, Stockport, Cheshire.

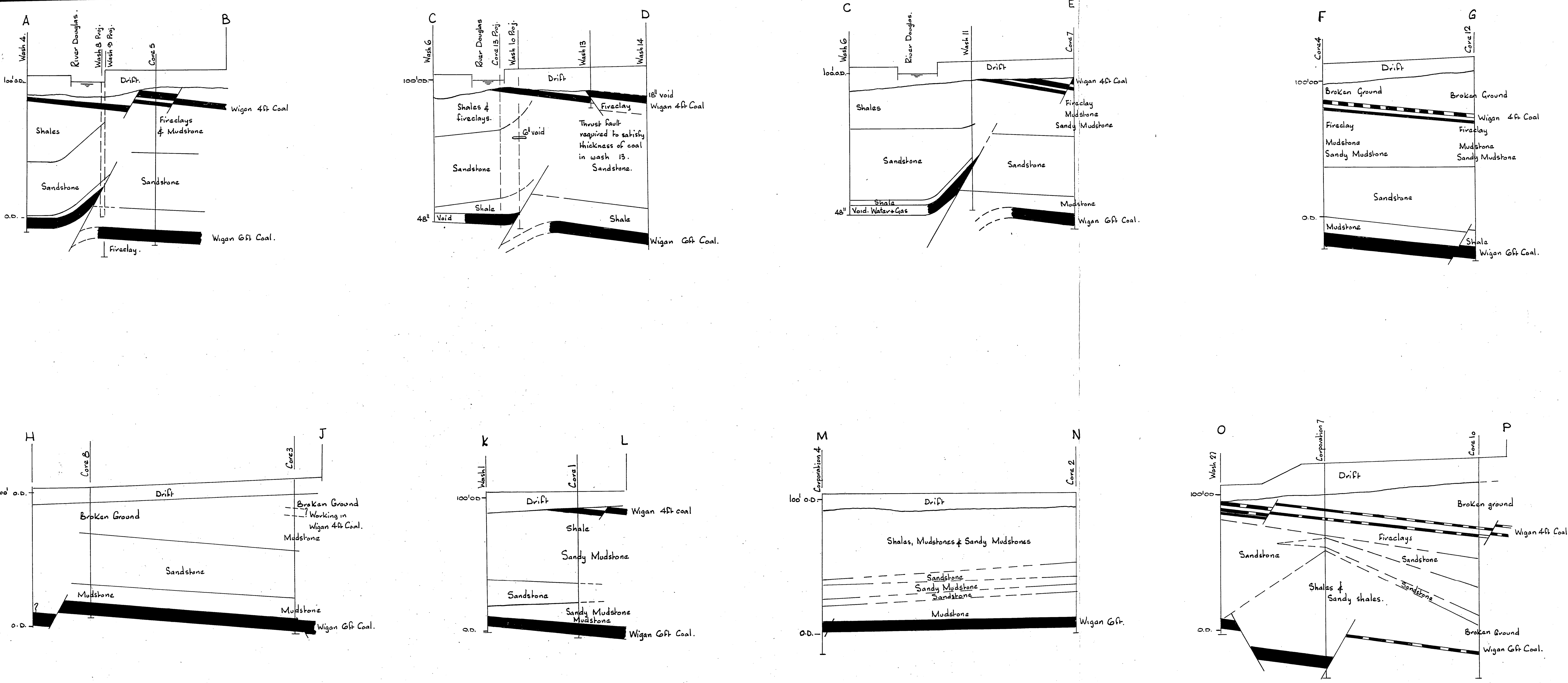
SITE INVESTIGATION
SITE PLAN

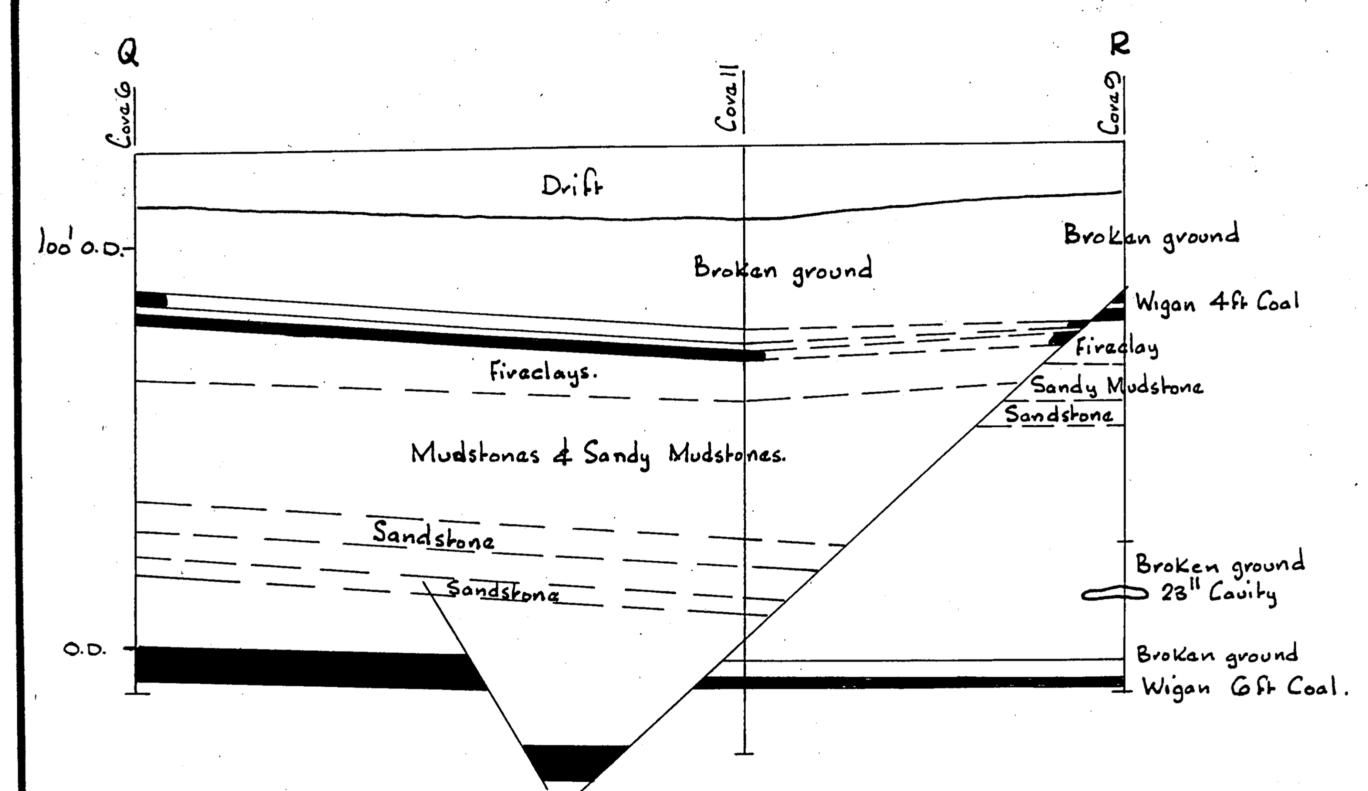
SCALE: 1 : 500th.

JOB No. SN859 DRAWING No. A

REVISION	
DATE	19-3-65
SIGNATURE	

30





Truscon LIMITED

50, Seymour Grove, Old Trafford, Manchester 16.
Telephone :- TRAfford Park 2766 & 3044.
Reg office :- 35-41 Lower Marsh, London S.E.1

WIGAN - Scholes Redevelopment
Multi-Storey Flats

G. Keighley B.Sc. A.M.I.C.E. M.I. Mun.E. Borough Engineer & Surveyor. Municipal Buildings, Wigan.	Main Contractors Rowlinson Construction. Sth. Reddish Stockport.

POSTULATED GEOLOGICAL
CROSS SECTIONS

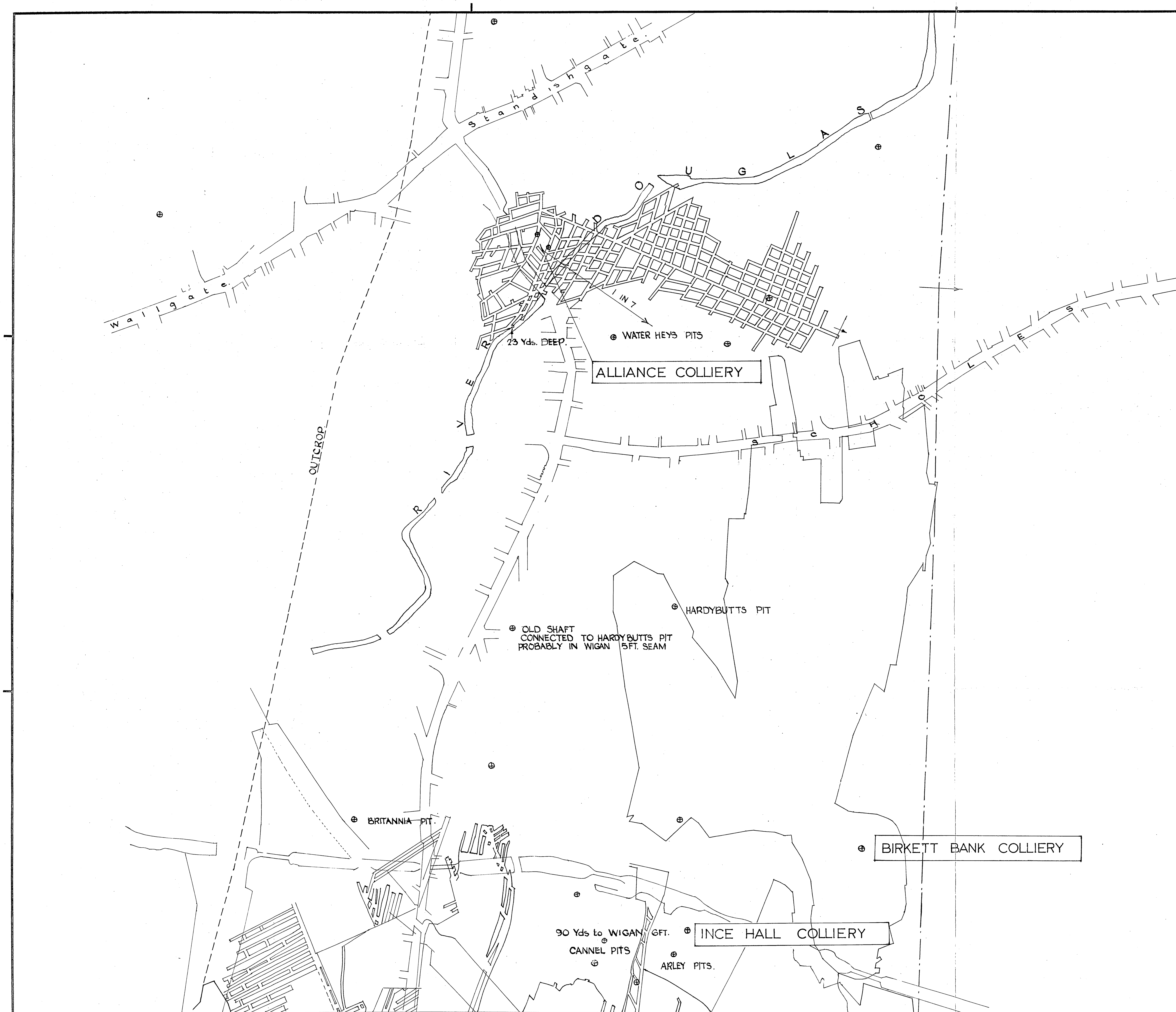
SCALE: 1/500 natural.

JOB No. SN 859 | DRAWING No. F

REVISION	
DATE	19-3-65
SIGNATURE	D. K.

33

LR 6818



Truscon LIMITED
50 Seymour Grove Old Trafford, Manchester 16.
Telephone: Trafford Park 2766 ; 3044.
Registered Office 35-41 Lower Marsh, London S.E.1

WIGAN - Scholes Redevelopment
Multi Storey Flats

G. Keighley B.Sc. A.M.I.C.E. M.I.Mun.E. Borough Engineer + Surveyor Municipal Buildings, Wigan.	Main Contractors Rowlinson Construction Ltd. Sth. Reddish, Stockport, Chesh.

TRACING FROM N.C.B. DRAWING INDICATING WORKING IN THE WIGAN 6FT. SEAM FROM ALLIANCE COLLIERY

SCALE:

JOB No. SN 859 | DRAWING No. G

REVISION		
DATE	19-3-65	
SIGNATURE		

31

LR 6818

TRUSCON LIMITED
50 Seymour Grove Old Trafford Manchester 16
Telephone Trafford Park 2766 3044
Reg Office 35-41 Lower Marsh London S.E 1

WIGAN - SCHOLES REDEVELOPMENT
MULTI-STOREY FLATS

Borough Engineer & Surveyor Municipal Buildings Wigan	MAIN CONTRACTORS

LOCATION OF BOREHOLES AND SECTION LINES

SCALE 1 500 th

JOB No. SN 859	DRAWING No. B

Revision					
Date	19-3-65				
Signature					

32

NOTES:

SN769 (Apr. 1964) - First Truscon site investigation — Boreholes shown thus — B.H.1.

Second " " " — " " " — B.H.4.

SN769 (Dec. 1964) " " " — Penetrometer tests shown thus — x P.6. (Located mainly in area of original deep excavation).

" " " — Rotary holes shown thus — R.7.

SN769A (Jan. 1965) - Third " " " — Bore holes " " — B.H.30.

SN934 (May 1965) - Fourth " " " — Penetrometer tests " — X P6 (Located outside the original deep excavation and adjacent to walls A, B, C, H & J.

SN 934 (May 1965) - Fifth " " " Further Penetrometer tests. Frankipile Ltd. compaction points CP1 to CP37 (excluding CP3) shown thus — CP4

SN 934 (July 1965) - Sixth " " " Further Penetrometer tests. Further compaction points (CP 38 to CP39) made by Frankipile.

Truscon LIMITED
Southmoor Road, Wythenshawe, Manchester 23
Telephone Didsbury 7771 6
Reg. Office. 35-41 Lower Marsh, London S.E.1

NORWICH
MALTHOUSE RD. CAR PARK

H.C. Rowley Esq. M.I.C.E., M.I.Mun.E.,
City Engineer, City Hall, Norwich.

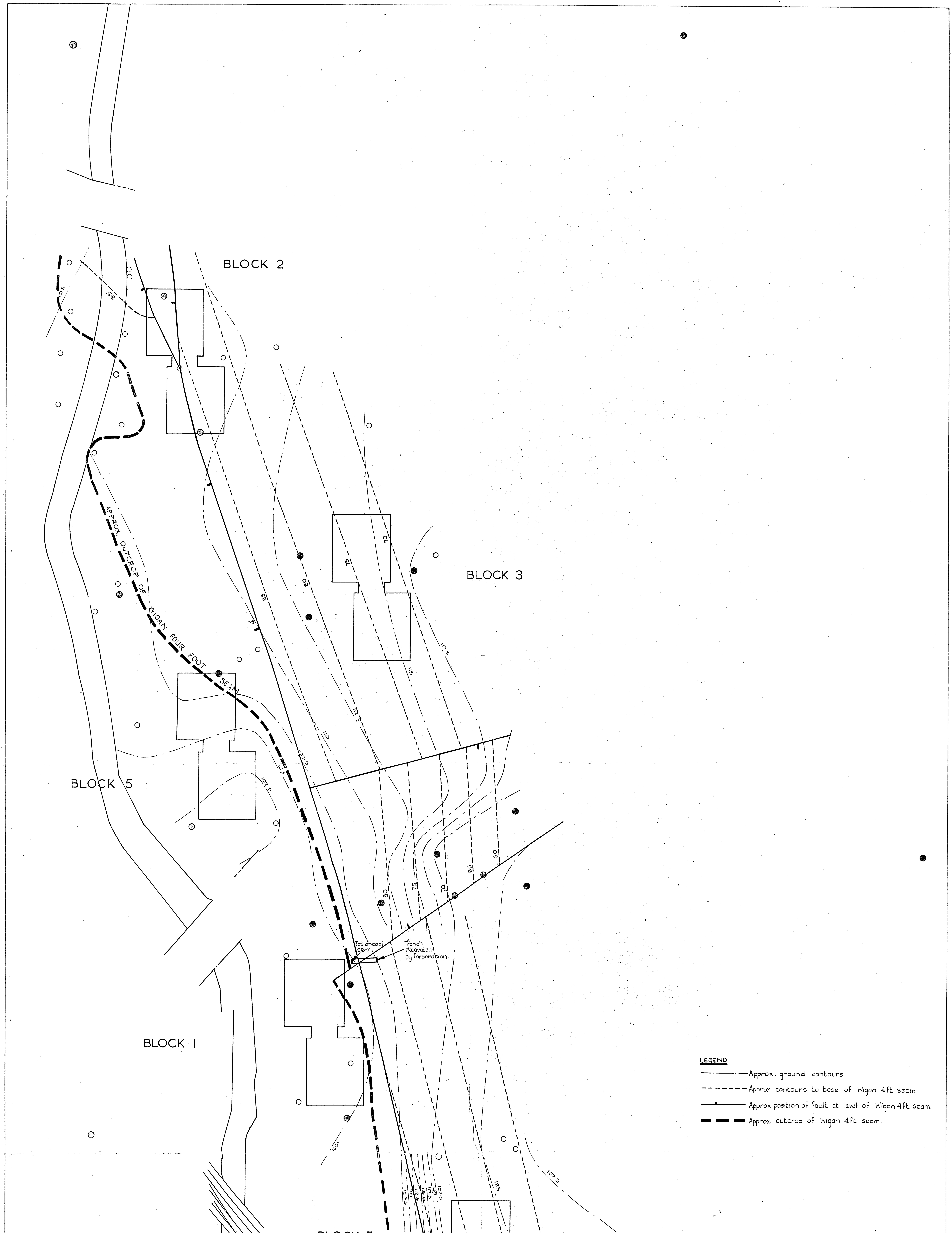
SITE INVESTIGATION
SITE PLAN.

SCALE: 3/16" TO 1'-0"

JOB No. **SN 934** DRAWING No. A

REVISION		1	2	3
DATE	17.12.64.	29.1.65.	May 65	May July 65
SIGNATURE	CJ	MT.	D. Riley	D. Riley

17



TRUSCON LIMITED
50 Seymour Grove, Old Trafford, Manchester 16.
Telephone: Trafford Park: 2766, 3044.
Registered Office: 35-41 Lower Marsh, London S.E.1.

WIGAN - SCHOLES REDEVELOPMENT
MULTI STOREY FLATS

G. Keighley B.Sc. A.M.I.C.E. M.I. Mun. E Borough Engineer + Surveyor, Municipal Buildings, Wigan.	Main Contractors. Rowlinson Construction Limited, South Reddish, Stockport, Cheshire.

APPROX. GROUND CONTOURS AND CONTOURS TO BASE OF WIGAN 4FT SEAM.

SCALE 1: 500 th.

JOB No. SN 859 | DRAWING No. D

Revision					
Date	19-3-65				
Signature	R. 3 Lowes.				

35


BLOCK 2
Approx. line of disturbance.
BLOCK 3
BLOCK 5
BLOCK 1
BLOCK 5
LEGEND
Approx ground contours.
Approx contours to base of Wigan 6Ft seam.
Approx. position of fault at level of Wigan 6Ft seam.
Approx. line of disturbance.
TRUSCON LIMITED
50 Seymour Grove, Old Trafford, Manchester 16.
Telephone: Trafford Park: 2766, 3044.
Registered Office: 35-41 Lower Marsh, London S.E.1.
WIGAN - SCHOLES REDEVELOPMENT
MULTI STOREY FLATS
G. Keighley B.Sc. A.M.I.C.E. M.I.Mun.E
Borough Engineer + Surveyor,
Municipal Buildings, Wigan.
Main Contractors.
Rowlinson Construction Limited,
South Reddish, Stockport, Cheshire.
APPROX GROUND CONTOURS AND CONTOURS TO BASE OF WIGAN 6 FT. SEAM.
SCALE 1:500 th.
JOB No. SN 859
DRAWING No. E
Revision
Date 19-3-65
Signature R. J lowes.
36

LEGEND.

Approx ground contour

Approx. contours to top of Coal Measures Strata.

TRUSCON LIMITED
50 Seymour Grove, Old Trafford, Manchester 16.
Telephone: Trafford Park: 2766, 3044.
Registered Office: 35-41 Lower Marsh, London S.E.1.

WIGAN - SCHOLES REDEVELOPMENT
MULTI STOREY FLATS

G. Keighley B.Sc. A.M.I.C.E. M.I.Mun.E. Borough Engineer + Surveyor, Municipal Buildings, Wigan	Main Contractors. Rowlinson Construction Limited, South Reddish, Stockport, Cheshire.

APPROX GROUND CONTOURS AND
TOP OF COAL MEASURES STRATA.

SCALE 1:500 th.

JOB No. SN 859 DRAWING No. C

Revision					
Date	19-3-65				
Signature	R. Flowers.				

34

TABLE 2

WIGAN - SN859. Occurance of Voids & 'Broken Ground'. TRUSCON LTD